MODEL TESTS ON REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

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By

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TO
MY
PARENTS

'Model Tests on Reinforced

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under my supervision and this work has not been submitted elsewhere for the award of a degree.

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ABSTRACT

MODEL TESTS ON REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

In this study laboratory model tests have been conducted on soft soil, granular fill, soft soil with unreinforced granular fill and soft soil with reinforced granular fill to study their load-settlement response. The IIT Kanpur campus soil is used as soft soil subgrade. The sand used as granular fill is poorly graded fine Ganga sand. Staple non-woven geotextile is used as reinforcing material.

Model tests are conducted in a cylindrical tank of diameter 40 cm and height 32.5 cm. Circular footing of diameter 7.5 cm is used in the test. Load is applied through a loading platform. The relative density (D_r) of granular fill is kept constant at 56.17% in all the tests. The required density is achieved by pouring the sand from a constant height using rainfall technique. The tests are conducted at two different density of soft soil subgrade. Loads are applied in small increments and each load is kept for 24 hours to achieve full consolidation to occur under that load. Reinforcement is provided at 1/3rd, ½ and 2/3rd depth in the granular fill.

The result of model test indicates that the load capacity of the soft soil subgrade is lowest followed by that of granular fill and the soft soil subgrade with unreinforced granular fill. The load capacity of soft soil reinforced with geotextile is maximum. Also, the load capacity first increases as the depth of reinforcement in the granular fill increases from 1/3D to ½ D and then decreases as the depth of reinforcement is further increased from ½ D to 2/3D. The maximum load capacity is achieved for the case when the reinforcement is provided at ½ D.

CONTENTS

Nomenclature

γ_d	Dry unit weight of sand
$\gamma_{d\,max}$	Dry unit weight of sand in the densest condition
$\gamma_{d\,min}$	Dry unit weight of sand in the loosest condition
σ	Normal Stress
σ_1	Maximum Principal Stress
σ_3	Minimum Principal Stress
τ	Shear Stress
ϕ	Angle of internal friction of sand
ϕ_{sg}	Friction angle between sand and geotextile
ν	Poisson Ratio
BPR	Bearing Pressure Ratio
B	Diameter of the geotextile
C_U	Uniformity Coefficient
C_C	Coefficient of curvature
D	Diameter of the footing
D_{10}	Effective diameter (Diameter at which 10% of the soil is finer)
D_{30}	Diameter at which 30% of the soil is finer
D_{60}	Diameter at which 60% of the soil is finer
D_{50}	Median diameter (Diameter at which 50% of the soil is finer)
D_r	Relative Density
$e_{max.}$	Void ratio of sand in the densest state
$e_{min.}$	Void ratio of sand in the loosest state
$e_{nat.}$	Void ratio of sand in the natural state
F	Final settlement of the footing
G_s	Specific gravity of soil
G_{gf}	Specific gravity of granular fill

I	Immediate settlement of the footing
K_0	Coefficient of lateral earth pressure at rest
K_s	Modulus of subgrade reaction of soil
K_{gf}	Modulus of subgrade reaction of granular fill
N	Number of the non-woven geotextile
O_{95}	Pore size opening at which 95% beads are finer
O_{90}	Pore size opening at which 90% beads are finer
O_{50}	Pore size opening at which 50% beads are finer
p	Stress path parameter $\{(\sigma_1 + \sigma_3)/2\}$
q	Stress path parameter $\{(\sigma_1 - \sigma_3)/2\}$
q_o	Bearing pressure of the soft soil
q_{as}	Bearing pressure of unreinforced granular fill-soft soil
q_{ars}	Bearing pressure of reinforced granular fill-soft soil
s	Settlement of the footing
u	Depth of first layer of reinforcement

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

There are different types of foundations, which can be provided below the buildings, storage tanks, car parks and other such lightweight structures. Generally, shallow foundations are provided for these types of structures, if the soil close to the ground surface possesses sufficient bearing capacity. However, in case the top soil is either loose or soft, the loads from the super structure has to be transferred to deeper firm strata, by providing pile foundations or the ground improvement technique may be adopted.

Now a days, due to the rapid growth in the population and in the industrialization there are shortage of good construction sites to provide shallow foundation. Also, the provision of pile or pier foundation is not suitable from the economy point of view in many cases. Reclaimed grounds are largely available now a days, for the construction of structures. These grounds are characterized by very low bearing capacity and undergo large settlements due to the application of the loads. Also, there are sites having very poor ground conditions, which will create problems for the civil engineers. These soils have very low bearing capacity (~ 7.0 kPa.) and would hardly support the weight of an individual.

In olden days, to get rid of such problems it was proposed to change the site, redesign the proposed structure, replace the ground with suitable soils, provide deep foundations, wait until natural consolidation occurs or attempt to modify the ground. The last or the ground improvement method is widely used now a days. There are several methods of ground

improvement e.g. mechanical modification, chemical modification, hydraulic modification etc., but the improvements by the use of geosynthetic is emerging as the best among all these methods and thus are widely used. When the geosynthetics are used for the purpose of ground improvement, they are mixed with the site soil and the combination is known as reinforced soil. The concept of the soil reinforcement is not new, since Roman times, builders have been aware of the beneficial effect of the reinforcing element in the earthwork. The Babylonians, more than 3000 years ago, used reinforced soil to build Ziggurats. In Asia also, from China to Japan, bamboo as well as straw have been utilised to strengthen earth as a building material.

In more recent times, engineers have reported on the use of fagots, composed of cuttings from tree branches for the stabilization of the riverbank. During past two- three decades geosynthetics are playing an important role in ground improvement. The first use of a woven synthetic fabric for erosion control was in 1950's in Florida by Barrett. The modern concept of soil reinforcement was proposed by Casagrande, who idealizes the problem in the form of a weak soil reinforced with a high strength membrane laid horizontally in layers (Westergaard,1939). Vidal (1969), a French architect and engineer gave the concept of reinforcing earth. He called reinforced earth a new kind of construction material created by the association of a particular medium with the reinforcement, which formed a "Volume having Cohesion".

The geosynthetic reinforced granular fill-soft soil systems are now being used very frequently as foundations for different structures like unpaved roads, shallow footings, low embankments etc. Such soil systems provide improved bearing capacity and reduced settlements, by distributing loads over wide area of the weak sub soil. In conventional methods, without the use of any reinforcement, a thick layer of granular fill was needed which may be proved costly or may not be possible, specially on the sites where there is a limited availability of good quality granular materials. Also, the use of soil reinforcement provides many benefits like speedy construction and good aesthetic views etc.

1.2 SCOPE AND ORGANISATION OF THE PRESENT WORK

A review of the available literature, related to the surface reinforced foundation soil has been presented in chapter 2. From the review of literature, it has been observed that the analysis of a foundation placed on geosynthetic-reinforced granular fill-soft soil system is important in regards to its settlement behavior under different load intensities and the parameters of the soil over which it rests. The prestressing of the geosynthetic reinforcement, the compressibility of the granular fill and the time-dependent behavior resulting from consolidation of the soft foundation soil, which need their considerations while using the geosynthetic-reinforced granular fill soft-soil system in the field and estimating its settlement under the applied load.

Chapter 3 deals with the details of materials used, arrangement of the model test, the test procedure and types of the tests conducted. The chapter also contains the line diagram of the model test setup and the modes of failure of reinforced soil under the application of loads. The results and discussion are given in chapter 4. Chapter 5 presents the conclusions drawn from the present work and the suggestions for the future work. Lastly a list of references is presented.

CHAPTER 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

The study of soil reinforcement is a recent topic. Though soil reinforcement was used since ancient times, no rational studies had been done at that time to predict the engineering behavior of reinforced soil. The reinforcing materials used at those times were natural tree branches, rope fibers, bamboo strips etc. The modern concept of soil reinforcement was proposed by Casagrande, who analyzed the problem as a weak soil reinforced with high strength members laid horizontally in layers.

Vidal (1969) proposed the modern concept of soil reinforcement. The approach led to rapid development. Much fundamental work was sponsored by various national bodies, notably at the Laboratoire des ponts et Chaussees (LCPC) in France (Schlosser,1977), by the United States Department of Transportation (Walkinshaw,1975) and by the United Kingdom Department of Transport (Murry,1977). These works led to a better understanding of the fundamental concepts involved and the introduction of improved forms of reinforcements such as grids, first used by the California Department of Transportation (Forsyth,1978).

There were various theories to explain the reinforcing mechanism. Various design methods were proposed and postulated with the assessment of bearing capacity of the foundation soil below the aggregate layer for the estimation of the surface deformation under the applied load. A number of finite element analysis were also carried out to study various parameters and the same, provide valuable results. This chapter presents a review of literature on the topic comprising of experimental, analytical and theoretical studies carried out so far.

2.2 REINFORCING MECHANISM

A number of concepts have been published to define the basic mechanism of reinforced soil. The essential phenomenon in reinforced soil is the friction, developed between the soil and the reinforcement. These frictional forces help the soil to transmit the stresses to the reinforcement. The reinforcement should be placed in the direction of tension and then the only interaction between the soil and reinforcement will generate necessary frictional forces. The higher the friction between geosynthetic and soil, more effective is the reinforcement. Thus an ideally rough bar, strip or sheet is significantly better than a reinforcement with smooth surface, (Schlosser and Elias, 1978).

The main concern in the reinforced earth is the interaction of the stress and the displacement between the soil and the reinforcement. The primary resisting force developed in the reinforcement, due to the mobilization of the frictional force along the entire length of inclusion, is the tensile force. This is known as "Membrane Effect", (Ghosh and Madhav, 1994). Discrete element analysis of reinforced beds have been conducted and it was observed that the "Confinement Effect" was derived from the interface shear stress imparted by the reinforcement which remains in tension under the applied load/ deformation,(Ghosh, 1991).

Experimental studies has been carried out in the tri-axial apparatus on sand samples. The samples were reinforced by horizontal thin aluminum plates (Schlosser and Long, 1972), and by uniformly spaced horizontal nets of fiber glass (Yang, 1972). In all the cases failure occurred either by excessive lateral deformation due to sliding of the sand on the reinforcement or by the breakage of the reinforcements. Schlosser and Long (1972) showed that the reinforcements provide to the sand an anisotropic cohesion directly proportional to their resistance to tension. From the results they interpreted the strength envelope for reinforced sand at failure by the breakage of the reinforcement, same as that

of a cohesive frictional Mohr-Coulomb material. They also showed that under small axial deformation, there is a rapid mobilization of the apparent cohesion induced by the reinforcements, while the mobilization of the internal friction angle is practically not affected by the presence of the reinforcements. Yang (1972) hypothesized that the tensile stresses built up in the reinforcements were transferred to the soil through sliding friction. This causes an increase in the confining pressure.

Housmann (1976) interpreted the effect of reinforcing the sand on its strength characteristics considering a global apparent friction angle (ϕ_R). He assumed that when failure is caused by slip between the sand and the reinforcement, the reinforcing effect could be expressed in terms of an increased apparent friction angle. It was suggested that the behavior of the reinforced soil system using extensible reinforcements does not fall within the concepts presented by Vidal (1969) for the reinforced earth and therefore, was termed as ply-soil by McGown and Andrews (1977). The ply-soil, e.g. the geosynthetic reinforced soil has greater extensibility and smaller losses of post peak strength as compared to the sand stones alone or reinforced earth.

The role of geosynthetic reinforcement in improving the load carrying capacity and settlement characteristics of the geosynthetic-reinforced foundation soils can be understood in the following manners. The increase of the subgrade bearing capacity by changing the failure mode; i.e. geosynthetics tend to force a general, rather than a local failure. The second is the reduction of the maximum applied stress due to a redistribution of the applied surface load below the geosynthetics by providing restraint of the granular fill if embedded in it or by providing restraint of the granular fill and soft soil, if placed at their interface (slab effect or confinement effect, Giroud et al., 1984; Madhav and Poorooshasb, 1989; Sellmeijer, 1990; Housmann, 1990). The third is the supplementary support due to membrane effect, i.e. the deformed geotextile provides an equivalent vertical support (Giroud and Noiray, 1981; Bourdeau, et al., 1982; Sellmeijer et al., 1982; Love et al., 1987; Madhav and Poorooshasb, 1988; Bourdeau, 1989; Sellmeijer, 1990; Housmann, 1990).

Geosynthetics improve the performance by acting as a separator between the soft soil and the granular fill known as the separation effect. Nishida and Nishigata (1994) studied the separation function of the geotextiles. Geogrids proves to be much effective because the soil particles get locked in the apertures of the grid membrane. This is known as anchoring effect (Giroud et al., 1986).

2.3 BEARING CAPACITY ANALYSIS

2.3.1 ANALYTICAL STUDIES

Binquet and Lee (1975) analyzed the problem of the bearing capacity of a strip footing with the horizontal layer of reinforcement in the granular fill. Based on the observed model they suggested three modes of failure of the reinforced slabs. These are (i) failure of soil above the top reinforcement, (ii) pullout of the ties due to slip and (iii) tensile failure of the ties. Using elastic approach they identified that maximum shear stress along the length of the reinforcement is a function of the depth of the reinforcement. They suggested that the maximum shear stress as 0.3 q_o at a depth of 0.2B and 0.1 q_o at a depth of 2B, where q_o is the vertical loading intensity on the footing.

Brown and Poulos (1981) demonstrated to use a finite element model of reinforced earth used to investigate the increase in the bearing capacity and stiffness of the foundation due to the placement of reinforcement in the soil. The finite element analysis was used to examine the effect of reinforcement on the load-settlement behavior of a strip foundation. It was shown that the improvement in the foundation performance depends on both the number of reinforcing layers and on the concentration (surface area per unit width of footing) of the reinforcement.

Giroud and Noiray (1981) developed a method for the design of geosynthetic reinforced granular fill-soft soil systems used as unpaved roads. They consider only the reinforcement action of geotextiles. The results were presented in the forms of charts established using the combination of (i) formulas relating to aggregate thickness and

traffic for unpaved roads without geotextile and (ii) a quasi static analysis comparing unpaved roads behavior with and without geotextile. This analysis showed the reduction in the aggregate thickness resulting from the use of geotextile. The method presented was developed on the reinforcement role of geotextiles and therefore, did not consider other beneficial effects of geotextiles such as separation, filtration and drainage.

Ingold and Miller (1982) present a theory to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and reinforced wall. The theory was based on the concepts that the reinforced embedded in the clay can be assumed to impart an equivalent undrained shear-strength to the clay. A series of model tests were also conducted to verify test data with that obtained by theory. The comparisons, showed sufficiently reasonable agreement between the theoretical and test results.

Sawicki (1983) studied the plastic behavior of soil that was unidirectionally reinforced by the fibers. Two failure mechanism of reinforced earth has been determined. The first mechanism depends on the simultaneous plastic flow of both the soil and the reinforcement. In the second mechanism the reinforcement remains rigid but the soil become plastic. The rigid-plastic model of reinforced earth was developed and its application was given. His model test results were checked experimentally by Kulczykowski. The proposed model describes the basic features governing the mechanical behavior of the reinforced earth. Slippage and edge effects both played important roles in the analysis of reinforced earth.

Broms (1987) presented a method to stabilize very soft clay (mud) using woven geotextile and preloading. He showed that by constructing narrow berms on the geotextile, spread over the area to be stabilized, it is possible to place the fill required for preloading without exceeding the bearing capacity of the soil. He suggested that the fabric should be stretched as much as possible to limit the penetration required to develop necessary tension in the fabric before placing the stabilizing berms.

Love et al. (1987) conducted analytical studies of reinforcement of a layer of granular fill on the soft soil subgrade. A finite element program was developed to study the problem

in more detail. The subgrade was modeled as an elastic perfectly plastic material with limiting shear stress equal to undrained cohesion, c_u. The fill material was modeled as an elastic-frictional material overlying the Matsuoka yield criterion (Matsuoka, 1976). Reinforcement was modeled by three noded line elements and treated as rough so that any failure must occur in the soil elements adjacent to the reinforcement rather than at the interface. Yielding of the reinforcement was not considered. The results of the finite element analysis showed that in the absence of reinforcement, substantial shear stresses might be transmitted to the clay surface, reducing its ability to resist vertical loading.

Koga et al. (1988) carried out finite element analysis of geogrid reinforced soil for two cases (i) an embankment on soft soil and (ii) a footing. The results obtained from the analysis showed that the tensile stresses were reduced in the weak subsoil with the use of geogrids as reinforcement for the case of embankments and did not influence the settlement characteristic of the embankment. However, the maximum settlement was considerably reduced both in the case of surface and embedded footings. From the results it was concluded that the geogrids are better than strips for soil reinforcement.

Madhav and Poorooshasb (1988) proposed a 3-parameter mechanical model for the geosynthetic reinforced granular fill-soft soil system. In this model, the geosynthetic reinforcement was represented by a foundation model element using a rough membrane. The granular fill and the soft subgrade were idealized by Pasternak shear layer and a layer of Winkler springs respectively. The results at small displacement indicated the effect of granular fill to be more and significant than that of the membrane in reducing the settlements of the reinforced soft soil system.

Poran et al. (1989) used finite element analysis to evaluate the settlement of footing placed on geogrid-reinforced granular fill overlying soft clay subgrade. They used visco-plastic model for soils and visco-elastic membrane elements to model behavior of geogrid reinforcement. Based on the parametric study, the results indicated the effect of geogrid reinforcement for improvement of the load deformation behavior of such system. Design procedure presented is applicable for continuous and axi-symmetrical footings.

Bourdeau (1989) presented a model to assess tensile membrane action in a two-layer soil system reinforced by geotextile. The analysis was based on a two-dimensional plane strain model of the static equilibrium of an elastic membrane placed at the interface between a granular base and a compressible soil subgrade. The model incorporates Mohr-Coulomb failure criteria for the frictional interaction at the gravel-fabric interface, and allows for incomplete anchorage. The results showed that the performance of high tensile modulus fabrics was affected to a higher degree by an incomplete anchorage than low modulus fabric.

Poorooshasb (1989) presented a procedure to analyse geosynthetic reinforced granular fill-soft soil system by using transform functions. The analysis showed that the contribution of geosynthetic in supporting vertical load imposed by the footing was through the tensile stresses developed in the grid. These stresses are due to the change in the geometry of the grid due to deformation of the system and due to dilatation properties of the fill. With the increase in the overconsolidation ratio of the granular fill (through compaction), efficiency of the reinforced soil system (ratio of the load for a predetermined settlement of the geosynthetic reinforced soil to the corresponding value for a soil system in its natural state) increases.

Madhav and Poorooshasb (1989) showed the effect of membrane in increasing the confining stress in the granular material due to the increase in shear modulus (G) with a Pasternak type foundation model. Results of the analysis showed a significant reduction in the overall and differential settlements due to the increased shear moduli. It was pointed out that extending the reinforcement beyond twice the width of the footing on either side of the center of the footing has less effect on settlements within the loaded region.

Vokas and Stoll (1989) used describe the response of a horizontally layered elastic system with one or more reinforcing sheets located at any prescribed depth below the surface by using a continuum model. The analysis was based on equations for layered systems from the linear elastic theory. The effect of reinforcement was included by

specifying the inter-layer boundary conditions on the basis of an analysis similar to that used in the classical theory.

Madhav and Ghosh (1990) extended the model suggested by Madhav and Poorooshasb (1989) to take account of the non-linearity of the soft-soil. It was showed that a single layer of geosynthetic at the interface of the granular fill and the soft soil improves the load response, the improvement being more in case of very soft soil.

Ghosh (1991) incorporate the non-linearity of shear stress-shear strain response of granular fill along with the application of multiple layers of geosynthetic reinforcement in the earlier work carried out by Madhav and Poorooshasb (1989) and Madhav and Ghosh (1990). The results of the analysis showed that under large loads, assumption of horizontal shear stress transfer at the fill-geosynthetic interface for inclined orientation of the geosynthetic reinforcement in the model may give erroneous results. This will surely happened because under large load, vertical shear stress transfer may be significant at the fill-geosynthetic interface.

Poorooshasb (1991) analyzed a system composed of geosynthetic granular fill supported by a subgrade in which subsidence took place. The objective of the analysis was to investigate various factor e.g. the extent of ground subsidence, the degree of compaction of the fill and the deformation properties and strength of the geosynthetic upon the performance of the system. Assumptions made were that the reinforcement was placed between the granular fill and the subgrade and that it behaved as a linearly elastic material of constant tangent modulus. The reinforcement was treated as rough so that no slip took place at the interface between the fill and the geosynthetic. Void ratio of the granular material played an important role in the reinforced fill behavior. Less compacted fills were capable of bridging over larger size cavities.

Pitchumani (1992) presented an analytical model by using elastic continuum approach to study the interaction between the reinforcement and soil, and to predict the reduction in surface settlement due to reinforcement below loaded area. Mechanism considered was the shear and normal stress interaction. From the analysis of the results it was showed

that the normal stresses resulting from vertical stress interaction for the strip as well as sheet are much higher than the shear stresses resulting from the horizontal stress interaction.

Dixit and Mandal (1993) made attempts to determine the bearing capacity of geosynthetic reinforced soil by using variational method. The analysis did not try to determine the actual rupture surface and the normal stress distribution or to calculate the actual bearing capacity. Instead, the rupture surface and the normal stress distribution that satisfied the conditions of limiting equilibrium and yielded the lowest possible value for the applied load were established. The results showed that the shape of critical rupture surface was a log spiral. The shape of the critical rupture surface depends on both the cohesion and the angle of internal friction. But this approach was valid only for shallow foundations.

Sridharan et al. (1993) carried out some numerical work to analyze the soil beds reinforced with the horizontally embedded reinforcement. They used elastic analysis and assumed that some part of the load carried by the footing on a reinforced soil bed will be transferred to the soil directly and the rest was transferred by the reinforcement. The load directly transferred to the soil causes settlement. The boundary between the downward and outward moving zones is the vertical plane passing through the edge of the footing and the failure can occur either by tie failure or the friction mode. To verify their analysis they also perform some experiments on reinforced and unreinforced sand. Relative density of the sand was 85%, uniformity coefficient of 3.5 and the coefficient of curvature as 0.97. Size of the footing used was 152× 915mm. Reinforcement used was in the form of aluminum strips of thickness 0.54mm, width 25.4mm and having a length of 457mm. Reinforcement was placed in three layers at a vertical spacing of 38.1mm. Results showed that while computing the frictional strength of lower layers, that component of load, carried by upper layers has been assumed to be distributed uniformly beyond the loaded area and hence not available for the mobilization of the frictional strength.

Espinoza (1994) presented a general expression for evaluating the increase of bearing capacity due to membrane action based on strict equilibrium condition. The model represents a subgrade underneath the geotextile composed of soft soil and a base of cohesionless material. By making an appropriate normalization of the membrane support expression, it has been shown that whether the deformation of the geotextile is assumed to be parabolic or circular with variable and constant strain along the geotextile, the different expression for the membrane support gives similar results for small rutting factor. Models assuming the circular deformation of the geotextile give larger values of the membrane support as compared to the parabolic ones for larger rutting factor.

Ghosh and Madhav (1994) proposed a new model for a reinforced shallow foundation bed by incorporating the rough membrane element for a single layer of reinforcement. The earlier model was modified to include the mechanics of the membrane element together with the non-linear response of the soft soil and the granular fill. Formulations for the plane strain loading conditions were extended for a single layer reinforced granular fill-soft soil system. Parametric studies indicate that at larger deformations, the vertical load component of the tensile force in the reinforcement resists the applied load from acting directly over the soft soil underneath. The membrane effect was significant at low values of the shear stiffness of the granular fill. With the increased value of the soil-reinforcement friction coefficient the membrane effect improved the load settlement response. With the decreasing strength of the soft soil, the reinforcement developed more tension due to larger settlements beneath the footing. Thus the membrane effect improves the load-settlement response of the footing with increase of the width of the footing.

Ghosh and Madhav (1994) developed a simple mathematical model to account for the membrane effect of the reinforcement layer on the load settlement response of the reinforced granular fill-soft soil system. They took into account the non-linearity of shear stress-settlement response of the soil through a parabolic relation for the response of a Winkler spring. Similarly a hyperbolic relation for the shear stress response of the Pasternak shear layer is taken into account. Results indicated that the improvements in

the settlement behavior of the geosynthetic reinforced granular fill-soft soil system subjected to uniformly distributed strip loading was significant with respect to stiffness of granular fill, when the soil is softer, and with respect to interfacial friction, when the fill material is less stiff.

Shukla and Chandra (1994) presented a mechanical model for idealizing the settlement response of a geosynthetic reinforced compressible granular fill-soft soil system by representing each sub system by commonly used mechanical element such as stretched, rough, elastic membrane, Pasternak shear layer, Winkler springs and dash pot. Strip loading in plane stress condition was considered. From the test results they observed that the compressibility of the granular fill has an influence on the settlement response of the geosynthetic reinforced granular fill-soft soil system as long as the stiffness of the granular fill is less than approximately 50 times that of the soft soil. Reduction in the settlement occurs due to both the prestressing of reinforcement and compaction of the granular fill.

Otani et al. (1994) used rigid plastic finite element method to study the bearing capacity analysis of geogrid reinforced foundation ground. To take account of the reinforcing effect in the analysis, a composite type model including geogrid and the surrounding soil was proposed. The results of the analysis showed that the bearing capacity of geogrid foundation ground increases as depth and length of the reinforcement were increased, but there existed an optimum depth in order to mobilize the maximum reinforcing effect.

Michalowski and Shi (1995) carried out analytical work to study the bearing capacity of the strip footings over two layer foundation soils. They used limit analysis to calculate the average limit pressure beneath the footings. The approach gives the upper bound to the true limit loads. On the basis of their studies, they presented design charts for the case, when the granular soil layer lies over the cohesive soil, either weak or strong. Analysis shows that the depth of collapse mechanism was found to be dependent on the strength of the clay. They proved that very weak clay attract the mechanism even at great depths.

Kaniraj (1995) carried out some analytical work to study the bearing capacity of non-homogeneous clay layer under the embankments. He used the properties of the Almere test embankment (Rowe & Soderman, 1984) to carry out the analytical studies. In his study Kaniraj ignores the berms, the offset for the trench from the embankments and the water present in the trench. The depth of the soft soil was taken as 3.75m. The embankment soil was considered as purely cohesive having undrained strength of 5.0kPa, instead of a granular soil with angle of internal friction ϕ equal to 33°. The result of the analysis shows, that inspite of the differences in the data used in the analysis, failure height and the reinforcement force have been found out to be nearly same, to the observed value by the authors. They also show that in both reinforced and unreinforced embankments, the rotational stability rather than the bearing capacity would govern failure, when the embankment width is larger than the thickness of the foundation soil.

Raghvendra et al. (1996) carried out finite element analysis by using elastic theory to study the bearing capacity of reinforced two layered soil system. They extended h approach given by Binquet and Lee for the design of reinforced soil bed as two layered system. They used four noded quadrilateral elements for the purpose of analysis of the problem. Final mesh size chosen has a dimension of 10B× 5B, where B is the width of the footing. From the analysis of two layered soil system below a strip footing, it has been shown that the vertical normal stress distribution in the upper layer was totally different from that of the semi-infinite soil system. It was also observed that the thickness of the upper layer plays an important role in the increase in bearing capacity of the two-layer soil system.

Wagh (1997) carried out nonlinear elastic analysis by using finite element method, to assess the optimum depth of placement for the single horizontal layer of geosynthetics. He carried out the analysis in two parts, in first part the reinforcement was treated as a thin flexible membrane incapable of taking in-plane compression, and in the second part the geotextile was treated as a thick sheet capable of taking in plane compression. Four noded iso-parametric continuum element were used to discretise saturated clay medium.

Interface between saturated clay and the reinforcement was discretized by using friction element as suggested by Verma et al. (1977). Nonlinear stress dependent behavior of clay was modeled by using hyperbolic stress strain relationship with Mohr-Coulomb failure criterion. Results show that the optimum depth of placement of thin single layer of geotextile is 0.375 times the diameter of the footing while the single horizontal layer of thick reinforcement was most effective when placed at the surface in reducing the settlement. The thin membrane reinforcement only resists the in-plane tension, which develops due to the relative movement and shearing stress at the interface.

Shukla and Chandra (1997) carried out numerical model studies to find out the effect of densification of granular fill in terms of increase in the lateral stress ratio on the settlement behavior of the geosynthetic reinforced granular fill-soft soil system. The model used was earlier proposed by Shukla and Chandra (1994). From the results of the model studies it can be concluded that the settlement of geosynthetic reinforced granular fill-soft soil system can be significantly reduced by increasing the lateral stress ratio. Also for lower shear parameter of the granular fill as well as for higher interfacial friction coefficients, the settlement of the reinforced granular fill-soft soil system reduces significantly with the increase in the lateral stress ratio of the granular fill. If the geosynthetic was prestressed then the effect of increase of lateral stress ratio in settlement reduction was reduced significantly.

Shukla and Chandra (1998) carried out analytical work to study the time dependent behavior of axisymmetrically loaded reinforced granular fill-soft soil system. In the model they proposed the geosynthetic reinforcement and granular fill were represented by stretched rough elastic membrane and the Pasternak shear layer respectively. The compressibility of the granular fill was represented by a layer of Winkler springs attached at the bottom of Pasternak shear layer. The soft soil subgrade was idealized as Terzaghi one dimensional consolidation model with dashpot and springs. The results of the analysis shows that the suggested model approach may be adopted to estimate the settlement of geosynthetic reinforced granular fill-soft soil system under axisymmetric

loading at any stage of consolidation. Horizontal stresses developed in the granular fill which leads to the reduction in the settlement. Prestressing of the geosynthetic reinforcement results in the reduction of both the total and the differential settlement. The compressibility of the granular fill has an appreciable influence on the settlement response as long as the stiffness of the granular fill was less than approximately 50 times that of the soil.

Kurian et al. (1997) carried out analytical work to study the settlement of the reinforced sand foundations. They perform detailed analysis on soil, reinforcement and the interface between the two. They used three-dimensional non-linear finite element analysis for their study purpose. They modeled soil as a three-dimensional isoparametric eight noded brick element and a three-dimensional truss element was used to model the reinforcement. The stress-strain behavior of the soil was approximated by the hyperbolic relationship. They used incremental or piecewise linear method for the analysis purposes. With the help of these they modeled the frictional behavior of the soil-reinforcement interface. The result of the analysis shows that the axial force in the reinforcement was found to be much less than near the center. The settlement of the reinforced soil system was found to be much less than that of the unreinforced case. The analysis also showed that the proposed three-dimensional analysis leads to a better understanding of the overall behavior of the reinforced soil under load.

Palmeira et al. (1998) carried out some back analysis of geosynthetic reinforced embankments on soft soils. They used two slip circle methods and an analytical method to estimate the safety factor of six trial embankments at their trial heights. Predicted values of the safety factors at failure heights were close to one, and in spite of limited data available the results obtained suggest that these simple methods are useful tools for predicting factor of safety of reinforced embankments when the required input data are available and accurate. They showed that the geosynthetic reinforcement can be effectively used to increase the factor of safety of embankments on soft soils.

2.3.2 EXPERIMENTAL WORKS

Binquet and Lee (1975) carried out model tests with strip footing on reinforced sand foundations. The dimension of the sandbox was 1.5m × 0.51m × 0.31m, the rigid footing was of 76mm width. The reinforcing materials consisted of 13mm wide strips of aluminum foils. Three series of tests were conducted; Series A tests simulate a deep homogenous sand foundation with long horizontal reinforcing strips. Series B test simulate a deep soft layer of clay or peat. This was done in the model by a layer of foam rubber. Series C simulate the reinforced granular soil over a soft pocket of material. The dry density and relative density of the soil used was 1500kg/m^3 and 75% respectively. Results of these tests were relatively consistent in showing that the load settlement and ultimate bearing capacity of the footings could be improved by a factor of about 2-4 times above the same load settlement or bearing capacity of an unreinforced soil for otherwise identical conditions. Tie pullout failure generally occurred with lightly reinforced slabs, N<2 or 3, whereas tie breaking, which occurred in the uppermost layers, was generally associated with heavy reinforced slabs, N>4.

Akinmusuru and Akinbolande (1981) performed laboratory tests with a square footing on a deep homogenous sand bed reinforced with flat strips of rope fiber material. The dimension of the box was 1.0m in width and 0.7m in depth. The square footing was of 100mm width and 13mm in thickness. It was observed that the BCR decreases with the horizontal spacing between the fibers and the rate of fall was higher for 0.5< x/B <1, where x is the horizontal spacing between the fibers. Also BCR increases up to N=3 after which there is very little contribution with increase in N.

Kinney (1982) performed small-scale laboratory tests to study the effect of geotextile on a soil-geotextile-aggregate system used as unsurfaced roads. The test demonstrated that a geotextile disc slightly larger than the loading plate had a very significant reinforcing effect on the reinforced soil-system; whereas, a disc identical in size to the loading plate did not appear to have any effect.

Mcgown et al. (1983) performed model tests on a strip footing for both unreinforced and reinforced (single layer) soil. The dimension of the test box was 2m × 3m × 0.94m and model footing was 0.3m long and 0.12m wide. The soil used was dense sand. They observed that the bearing capacity was greatly influenced by the location of the geotextile. If it is placed closed to zero extension line, a weakening of the system occurs due to the development of the failure surface along the geotextile and if it is placed along the direction of the principal strain, it leads to an overall strengthening of the system.

Fragaszy and Lawton (1984) studied the effect of soil density for a wide range for relative densities (D_r = 51% - 90%) and reinforcing strip length on the load settlement behaviour of reinforced sand by carrying out laboratory model tests. A rectangular steel footing 76mm wide by 152mm long was used for the test. The 56cm × 122cm rectangular test box was filled with 36cm deep sand. The density at which the tests were done are respectively 1.47, 1.54, 1.59 Mg/m^3 (D_r = 45%, 70% and 90%). In all the tests, the layer of reinforcement was located at depths of 2.54, 5.08 and 7.62 cm below the footing. The result indicated that when bearing capacity ratio was calculated at a settlement equal of 10% of the footing width, the bearing capacity ratio was independent of soil density. When calculated at a settlement of 4% of the footing width, the percent increase in the bearing capacity appeared to be less for loose sands than for dense sands. As strip length increased from 3-7 times the footing width, the bearing capacity ratio increased rapidly.

Guido et al. (1985) carried out laboratory tests to investigate the bearing capacity of a square footing on a geotextile reinforced sand of medium compact density (relative density =50%). The size of test box was 1.22m square with a depth of 0.92m. Test were carried out on two types of soils, first was having a dry density 14.80kN/m^3 and a relative density of 50%, the second test was on the soil of density 14.26kN/m^3 and a relative density of 50%. The square foundation size was 0.31m wide. All the test was conducted with square sheet of reinforcement. The load settlement response for unreinforced soil showed general failure mode while reinforced soil showed local shear (punching) failure

mode. At lower s/B ratio the unreinforced soil was stiffer than the reinforced soil. The location of the top layer of reinforcement has an effect on the bearing capacity ratio.

Love et al. (1987) performed model tests to study the effectiveness of the geogrid reinforcement, placed at the base of a layer of granular fill on the surface of soft clay. The size of the box was 1000mm × 300mm × 600mm. The width of the footing was 75mm. Monotonic loading was applied by a rigid footing, under plain strain conditions. From the results they concluded that the bearing capacity increases with the thickness of the fill. The reinforced system performed better than the unreinforced one. The surface heave was found to be less for reinforced system. For reinforced system compared to unreinforced system, the failure planes extends more deeply in to the subgrade and the heave profile at the surface and at the fill-clay interface is smoother and extends over a greater distance laterally.

Sakti and Das (1987) investigated the ultimate bearing capacity of a model strip foundation resting on saturated soft clay reinforced with geotextile layers. It was found that the geotextile layers placed under the foundation within a depth equal to the width of the foundation had some influence on the increase of the short-term ultimate bearing capacity. It was suggested that for maximum efficiency, the first layer of geotextile should be placed at a depth of about 0.4 times the width of the foundation and the minimum length of the reinforcing geotextile layers for maximum efficiency was about four times the width of the foundation.

Das (1989) presented laboratory test results for ultimate bearing capacity of strip and square shallow foundations supported by a compact sand layer underlain by a soft clay with or without a geotextile at the sand-clay interface. The results indicated that with the use of geotextile, the critical value of H/B ratio at which the maximum bearing capacity ratio occurred was about 0.75 for strip foundations and about 0.5 for square foundations, where B is the foundation width and H is the thickness of compacted sand layer below the base of the footing.

Venkatappa et al. (1990) carried out experimental studies to study the strength behavior of geotextile reinforced sand under axisymmetric loading. They carried out triaxial tests to assess the influence of the reinforcement on the overall behavior of the geotextile-reinforced sand. Study consists of conducting drained tests on sand specimen with and without reinforcement under different confining pressures. Sand used were Yamuna and Ottawa sand with relative density, uniformity coefficient and coefficient of curvature as 60% and 65%, 1.01 and 0.72, 0.83 and 0.92 respectively. Both the woven and non-woven geotextiles were used in the test. On the basis of the experiments they showed that the strength and stress-strain behavior of sand was improved by woven and non-woven geotextiles. c' and ϕ' values also increases up to a critical confining pressure of around 100kPa beyond which no appreciable increase were recorded. The strength ratio increases with increase in the number of reinforcing layers and decreases with increase in confining pressure.

Mandal and Sah (1993) carried out bearing capacity tests on model footings on clay subgrades reinforced with geogrids horizontally. Test results showed that the effectiveness of geogrid reinforcement increased the bearing capacity of clay subgrades, with improvements being observed at nearly all levels of deformation. Results indicated that the maximum percentage reduction in settlement with the use of geogrid reinforcement below the compacted and saturated clay was about 45% and it occurred at a distance of 0.25B (B being the footing width) from the base of square foundation.

Das and Shin (1994) carried out laboratory model tests to determine the permanent settlement of a surface strip foundation supported by geogrid-reinforced saturated clay and subjected to a low frequency cyclic loading. The foundation was initially subjected to an allowable static load. Based on the test results, it was concluded that for a given amplitude of cyclic load intensity, the maximum permanent settlement increased with increase in the intensity of static load, and for a given intensity of static loading, the maximum permanent settlement increases with increase in the amplitude of the cyclic load intensity. It was also pointed out that full depth geogrid reinforcement might reduce

the permanent settlement of a foundation by about 20% to 30% to one without reinforcement.

Floss and Gold (1994) examined the improvement of the bearing and deformation behavior by means of a geosynthetic reinforcement placed at the base of a layer of granular fill on the surface of soft clay (called reinforced two layer system). It was tried to present the reasons for improvement by carrying out both site tests and finite element analysis. It was pointed out that with reinforcement the granular layer was able to transmit shear forces on an essentially higher level without collapsing. The function of better load spreading was maintained also on a high deformation level.

Khing et al. (1994) presented a laboratory model test results for the ultimate bearing capacity of a surface strip foundation supported by a strong sand layer of limited thickness underlying by a weak clay with a layer of geogrid at the sand-clay interface. From the results it appeared that the optimum height of the strong sand layer should be about two-thirds that of the foundation width for obtaining the maximum benefit from the geogrid reinforcement in increasing the ultimate bearing capacity. These results were presented by conducting tests at one relative density of sand and one undrained shear strength, it will be more logical to verify results for several other sets of relative densities and undrained shear strength.

Nishida and Nishigata (1994) carried out laboratory tests by applying cyclic load on the surface of the pavement model in cylindrical mould to evaluate the separation function of the geotextile and to find out its relationship with reinforcement function. It was pointed out that the reinforcement was a prime function when the ratio of the applied stress on the subgrade soil to the shear strength of the subgrade soil (σ/c_u) was high, however, the separation could be an important function when the ratio was low.

Yetimoglu et al. (1994) investigated the bearing capacity of rectangular footings on geogrid-reinforced sand by performing laboratory tests as well as finite element analysis. Both the experimental and analytical studies indicated that there was an optimum reinforcement embedment depth at which the bearing capacity was highest when single-

layer reinforcement was used. Also, there appeared to be an optimum reinforcement spacing for multi-layer reinforced sand. The analysis, for the conditions investigated, indicated that increasing reinforcement stiffness beyond 100 kN/m would not bring about further increase in the bearing capacity.

Aiban and Znidarcic (1995) carried out centrifuge analysis to find out the effect of different stress levels and strain conditions on the granular materials. In their work they also studied the effect of load eccentricity, load inclination and critical burial on the bearing capacity of shallow strip footings on dense sand. The centrifuge has a capacity of 15g ton, radius of 1360mm to the base of swinging basket, which were 445mm wide. Each footing has U-shaped cross-section with longitudinal grooves to apply the loads at different eccentricities. Footings were made of Aluminum with sand glued at their bottom to get rough base. Footing has a width of 38mm, base thickness of 6mm and side wall thickness of 5mm. The central footing was 51mm long and each external footing were 38mm long. The sand used was U. S. fine silica sand with uniformity coefficient of 1.7, specific gravity of 2.65 and mean grain size of 0.18mm. It was pluviated from a height of 76.2cm to get a relative density of 88%; angle of internal friction of the sand was 42°. Results shows that under plane strain condition, the theoretical solutions overestimates the value of N_γ and thus the bearing capacity of the footings. Failure takes place in a progressive manner. The result of experiment agrees with the assumption of a linear pressure distribution at the bottom of the footing.

Fannin and Sigurdsson (1996) presented a field study of the construction, instrumentation and response of an unpaved road, stabilized with the geosynthetics. The site was on the right-of-way of highway 91/91A, near New Westminster, B.C. The soil encountered in the test section was mainly comprised of organic silts and clays, silts and sandy silts and peats in a layered manner up to a depth of 15m overlies a dense sand or silty sand. Vane shear tests indicate that the soil has undrained shear strength ranging between 45kPa at grade level to 30kPa at a depth of 15m. Sensitivity of the soil was found to be 7. The road section was divided in to five parts, having unreinforced section; three geosynthetic reinforced section and one section reinforced with geogrid. Three types of geosynthetics

were used having thickness of 1.8, 2.3 and 1.8mm, and burst strength of 2.4, 2.2 and 1.6MPa respectively. The geogrid used has a thickness of 0.8mm at the ribs and 2.8mm at the junctions. The aperture size of the geogrid was 25× 33mm. Test results shows that, considerable improvement were there in terms of trafficability, when geosynthetic was provided between the base course and the subgrade soil. Results also shows that for the thin layer of base course, geosynthetics perform better than the geogrids, in which case the separation effect was more predominant, while as the base course thickness increases, geogrids perform better than the geosynthetics. In this case, the reinforcement action becomes more dominant.

Murthy et al. (1996) carried out in-situ and laboratory tests to study the influence of geotextile as a reinforcing material in roads on soft subgrades. The objective of the work was to study the relative efficacy of the use of geosynthetic in the construction and maintenance of road pavement on soft soil subgrade. Three types of geotextiles used in the test are woven, non-woven and nonwoven needle fleeced. The geotextiles were placed between the subgrade and sub-base course. The soil in the test section has plasticity index of 30-40%, Proctor density of 1.50-1.60kg/cm^3 and the coefficient of uniformity varying from 8 to 10. The soil encountered in the test section was chiefly Black Cotton soil. Geotextiles used in the test have permeability value of 13×10^{-3}, 27×10^{-3} and 36×10^{-3} cm/sec. (at a pressure of 1kg/cm^2). O_{95} of the geotextiles are 0.090mm, 0.045mm and 0.030mm respectively for woven, nonwoven and nonwoven needle fleeced geotextiles. Results of the test show that the properly designed and installed geotextiles can increase the stability of the weak subgrade soils. The separation and confinement effect plus the strength gained by the frictional interlock between the aggregate and the fabric helps to reduce the stresses on the subgrade, thus increasing the load bearing capacity of the structural section and the soil subgrade. The geotextiles were also useful in dissipating the excess pore water pressure in to the granular materials of the base course. The results also shows that the nonwoven geotextiles were proved to be good as compared to the woven geotextiles due to their high percentage elongation and better filtration characteristics.

Ranjan, Vasan and Charan (1996) carried out a series of triaxial compression tests on cohessionless soils, reinforced with discrete randomly oriented fibers, to study the influence of fiber characteristics, soil characteristics, density and confining pressure on the shear strength of reinforced soils. On the basis of this they proposed a model to estimate the strength of the soils reinforced with any type of fibers and under any stress conditions. They used coir and plastic fibers having diameter of 0.2mm and 0.3mm, specific gravity equal to 0.75 and 0.92 and tensile strength of 1.0×10^5 and 1.5×10^5 kPa. Five different types of soil were used in the test, with all the soil were uniformly graded and having a uniformity coefficient ranges between 2.28 to 2.38. Soils used in the tests were fine sand, sandy silt, medium sand, silty sand and sand with average grain size diameter of 0.29, 0.058, 0.15, 0.55 and 0.45mm respectively. The cohesion value of these soils is 21, 31, 15, 24 and 18kPa respectively. Results of the tests indicate that the randomly distributed fiber reinforced soil samples exhibits higher residual strength as compared to the unreinforced case. The critical confining stress, below which the fibers tend to slip, increases with the decrease in the aspect ratio of the fibers. The increase in strength with increasing amount of fiber was linear up to 2% of fiber content.

Patel (1997) carried out model tests to study different factors controlling the reinforcement action of geosynthetics. Experiments were carried out on sand having relative density equal to 60%, D_{10} equal to 2.65mm, C_u equal to 2 and D_{50} equal to 0.48. Angle of internal friction of sand was 42° and the density of sand at which the experiment was carried out was $1.637 g/cm^3$. The geotextile was laid at a depth of 0.7B (width/dia.) of the footing below the sand surface. Its minimum extent was 2B beyond the edge of the footing so as to get the best improvement (Patel, 1981). Load tests were conducted by using a strip footing 90mm width and a circular footing of 5cm diameter. For the former a sand bed 20.3cm and 81.5cm in plan and 28cm thick was used and for the later sand bed of diameter 25cm and thickness 12.5cm was used with a surcharge pressure of $0.3 t/m^2$. Non-woven geosynthetics were used in the test with average density of $310 g/m^2$ and thickness varied from 0.43 to 1.56 mm. Results shows that for tension characteristic of geotextiles, wide strip tests and CBR push through test are reliable. Non-woven

geotextiles are not good for reinforcement purposes. Strains in woven geotextiles were large than in nonwoven geotextiles in pullout mode. In mobilizing the safe bearing capacity the contribution of interface friction was largest, tension contribution was very low and pullout frictional contribution was almost zero.

Chandra and Agarwal (1997) carried out experiments to study the effect of fiber reinforcement in improving the strength and bearing capacity of the clay. Unconfined compressive strength tests and triaxial compression test with unconsolidated undrained conditions were carried out on Kaolin soil with randomly distributed discrete polypropylene and jute fibers. Tests were conducted in a wooden frame of dimension 47× 20× 7.8 cm with Perspex sheet of thickness 6mm. and dimension 61.4× 27.2 cm was attached on each side of it to see the failure pattern. Fiber reinforced soil samples were prepared at a max. density of 1.5kg/cm^3 and OMC of 27.5%, obtained by conducting standard Proctor's test on undisturbed soil. Size of the footing was 2× 7.8cm. Results shows that the reinforced soils exhibits greater extensibility and no loss of post peak strength, as compared to unreinforced one. The secant modulus increases at all the levels of axial strains. Jute fibers are more effective in increasing the bearing capacity and strength.

Verma and Pandya (1997) carried out plate load tests to study the bearing capacity aspect of the geogrid reinforced foundation. The tests were conducted in a square tank of dimension 1.5m and a depth of 1.5m. The tank was filled with sand and the tests were conducted on sand at a relative density of 0% and 100% in both reinforced and unreinforced condition. Two types of geogrids were used, the first have a thickness of 3.1mm and tensile strength 9.23kg/cm while the other has thickness of 1.50mm and tensile strength equal to 4.0kg/cm. Angle of frictional resistance for these geogrids were 30° and 40° respectively. From the test results it can be concluded that the geogrid largely increase the ultimate bearing capacity when the sand was loose, also the settlements were considerably reduced. The increase in the bearing capacity was of the order of 450% in the loosest conditions due to the reinforcement.

Som and Sahu (1997) carried out a series of model tests to assess the effect of deformation on the improvement of reinforced soil bed. For the reinforcement purpose both woven and non woven geotextiles were used. The model tests were conducted in a tank of diameter 700mm with 150mm diameter footing. The footing rests on a compacted furnace bottom ash of thickness 40, 75, 110 and 150mm respectively overlying artificially consolidated Kaolinite with and without geotextile at the interface. The fly ash has a sp. gr. of 2.10, uniformity coefficient of 2.08 and an OMC of 29%. Angle of internal friction was 46° and a maximum density of $1.16gm/cm^3$. Kaolinite has a sp. gr. of 2.65, plasticity index of 20% and silt and clay in the ratio of 71 is to 29. The non-woven and woven geotextiles have a thickness and tensile strength of 3.40mm, 0.65mm and 3160kg/m, 4120kg/m respectively. Results shows that up to a settlement of about 10mm no improvement in settlement were observed with geosynthetic placed at interface of the fill soft soil subgrade. After this the rate of increase of settlement in the unreinforced soil bed was much higher than the reinforced case.

Nagraj (1997) carried out experimental work to find out the effects of horizontally placed geogrids on the bearing capacity and settlement stiffness of the sand foundation. The soil has a specific gravity of 2.65, natural dry unit weight of $16.3kN/m^3$ and a friction angle of 41°. Three types of geosynthetics were used in the tests namely CE111, CE 121 and CE131 having tensile strengths of 2.0kN/m, 7.68kN/m and 5.8kN/m and thickness of 2.8mm, 3.2mm and 5.5mm respectively. Test were conducted in a rectangular tank of size 81.5cm× 20.1cm×50cm. The rigid footing has a width of 8.9cm and 2.5 cm thick. Load was applied by hand operated screw jack. Tests were conducted by varying number of reinforcing layer from 1 to 5 keeping depth of top layer and vertical spacing between the layers constant. Results shows that the increase in the BCR and reduction in the settlement is a function of the tensile strength of the reinforcing layers.

Kurian et al. (1997) did some experimental work to study the settlement of reinforced sand foundations. Model tests were carried out on a sand bed in a tank. The dimension of the tank was 1.6× 1.6× 0.75m. They used steel footing having dimension 200× 200× 18mm. The soil used in the model test was uniformly graded sea sand, with specific

gravity of 2.72, D_{10} value of 0.23mm uniformity coefficient of 1.34 and minimum and maximum void ratio of 0.51 and 0.72 respectively. The sand was filled in layers of 100mm with top layer of 150mm. Total depth of sand layer was 750mm and each layer was compacted by a falling weight to get a relative density of 50%. Coir rope having 4.3mm diameter tied to bamboo strips of size 35× 5mm were used as reinforcing material. Four layers of reinforcement were placed at a vertical spacing of 100mm. Maximum load applied was 9kN. Test result shows that the settlement of the reinforced soil was much less than that of the unreinforced case, particularly at values close to the working loads. The axial forces in the reinforcement were maximum near the center and gradually reducing towards the ends.

Adams and Collins (1997) carried out large model tests on the sand reinforced with the geosynthetic. Total of 34 test were conducted to evaluate the effects of single and multiple layers of the geosynthetic layers placed below the shallow spread footings. Tests were conducted in a test pit having length 6.9m, width 5.4m and depth of 6.0m. Precast steel reinforced footings were used for the load tests. Four footings of sizes 0.3× 0.3, 0.46× 0.46, 0.61× 0.61 and 0.91× 0.91m were used in the test. Fine cement mortar sand was used for making the bed. The sand used was poorly graded with D_{50} value of 0.25 and a uniformity coefficient of 1.7. Types of geosynthetics used in the test are punched/drawn polypropylene bi-axial geogrid with aperture size 25× 30mm and ultimate strength equal to 34kN/m and the other geosynthetic was a geocell having a thickness of 1.25mm. Three types of tests were conducted, first without reinforcement, other with the use of geogrid and the last with the use of geocell. Results shows that, the geosynthetic reinforcement can substantially increase the ultimate bearing capacity of shallow spread footings. Three values of the geosynthetics give higher values of BCR as compared to one/two layer of reinforcement. Results also shows that the spread footings tested on reinforced sand foundations were less likely to experience general shear failure, plunging failure provided the first layer of the reinforcement was placed within 0.4B beneath the base of the footing.

Das et al.(1998) carried out some laboratory work to find out settlement of a square surface foundation supported by geogrid-reinforced sand and subjected to transient loading. The tests were conducted with one model foundation at one relative density of compaction using only one type of geogrid. Based on the model test results, it appears that geogrid reinforcement does reduce the settlement of the foundation. The effectiveness of geogrid reinforcement in reducing the settlement can be visualized by defining the term settlement reduction ratio R, or $R = \frac{S_{d(ult)-d}}{S_{d(ult)\ d-0}}$. Where $S_{d(ult)-d}$ = ultimate settlement due to the transient loading with reinforcement depth, d; and $S_{d(ult)-d=0}$ = ultimate transient loading settlement with no reinforcement. It is also clear from the experiments that the settlement reduction factor is a function of the depth of reinforcement.

Tan et al. (1998) did some experiments to evaluate residual shear strength of soft soils. In using geotextiles for geotechnical engineering applications, the soil-geotextile interface shear strength is an important design parameter. They used dry sand and non-woven polypropylene geotextile. The test result shows that the peak friction angle and residual friction angle of the sand-geotextile interface is not significantly affected by the nominal mass of geotextile. Result also shows that the interface peak and residual friction angle are not greatly influenced by the rate of shear. The result also shows that the interface friction angle decreases between the sand-geotextile, as the overburden pressure increases.

2.4 CONCLUSION

In the previous sections, a review of available literature related to the reinforced foundation soil was presented. From the results of a large number of model tests conducted and also from the results presented through several analytical and numerical studies of geosynthetic reinforced granular fill-soft soil system, it was observed that the geosynthetics, particularly geotextiles and geogrids show their beneficial effects as

reinforcements only after relatively large settlements, which may not be a desirable feature for shallow footings, paved and unpaved roads, embankments etc. Hence, the need has been felt for a technique, which can make the geosynthetic more beneficial without the occurrence of large settlements.

From the above review it is clear that, not much experimental work has been done to study the effect of geotextile on the load settlement response of the geosynthetic reinforced granular fill-soft soil system. Most of the earlier work has been done, only for the study of the interface shear strength and friction of the sand-geotextile composite. In this thesis work the load settlement response of reinforced granular fill-soft soil system has been observed and is presented. Model setup and material properties have been presented in next chapter.

CHAPTER 3

DETAILS OF EXPERIMENTAL STUDIES

3.1 INTRODUCTION

In this study various model tests have been conducted to study the effect of providing geotextile on the load-settlement response of reinforced granular fill-soft soil system. The details of materials used, model test arrangements, test procedures and types of tests conducted are presented in this chapter.

3.2 MATERIALS USED

3.2.1 SAND

Ganga sand is used for the model tests. The sand used for the testing is sieved through 1.0mm size sieve. Fully sun dried sand is used for the experiment. Various tests are conducted following standard testing procedures in the laboratory, to determine different properties of sand as given in Table 3.1. To keep the relative density constant, in each experiment a fixed weight of sand was allowed to fall from a constant height. Direct shear tests are conducted to calculate the angle of internal friction, Poisson's ratio and Shear modulus of sand.

3.2.2 SOIL

For testing purposes the soil used is taken from the site near the Aerospace department, in the I.I.T. campus. Various tests are carried out on the soil sample like determination of Atterberg limits, specific gravity, and grain size analysis etc. The soil is compacted in each test at the same density as closely as possible. The tests carried out on soil to determine its various properties are given in Table (3.2).

3.2.3 REINFORCEMENT

For the reinforcement purposes staple non-woven geotextile is used. The geotextiles are made of polypropylene. For calculating the ultimate strength of geotextile, strip tensile tests are conducted in ACMS, I.I.T. Kanpur. The size of strip used for the test was 25×100 mm. The test was done at a full-scale load of 0.2 kN, cross head speed of 10 mm/min, and a chart speed of 10 mm/min and 20mm/min. The types of tests carried out on geotextile are given in Table (3.3).

3.3 DIRECT SHEAR TEST

3.3.1 SHEAR PARAMETERS

The direct shear tests are conducted at three normal stresses of 50, 100 and 150 kN/m^2 on the Ganga sand. The size of shear box used was 6 × 6 cm. For all the tests the rate of loading was 0.25 mm/min. From these results peak angle of internal friction was calculated.

Table 3.1: Details of tests conducted on Sand

SN.	Types of test conducted	No of test
1.	Particle size distribution	1
2.	Specific Gravity	1
3.	Max. unit wt. Of sand	1
4.	Min. unit wt. Of sand	1
5.	Direct shear tests	3

Table 3.2: Details of tests conducted on Soil

SN.	Types of test conducted	No of test
1.	Grain size distribution	1
2.	Atterberg Limits	1
3.	Specific Gravity	1
4.	Consolidation test	1

Table 3.3: Details of tests conducted on Geotextile

SN.	Types of test conducted	No of test
1.	Thickness	1
2.	Tear Strength	1
3.	Weight	1
4.	Pore size diameter	1

3.4 MODEL TEST ARRANGEMENT AND TEST PROCEDURE

The model tests are carried out in a cylindrical tank, made up of G.I. sheet, having diameter 40 cm and height equal to 32.5 cm. The inside wall of the tank was smooth; hence the frictional effect would be small and may be neglected for all practical purposes. There are small holes at the bottom of the tank, to achieve proper drainage for water and also for circulation of air. The footing used for the test was circular, made up of steel, having a diameter of 7.5 cm and thickness of 1.0 cm. For the reinforcing material geo-textile was used.

The soil was collected from the site and then oven-dried and broken up in to fine material. All the lumps, stone parts and other organic parts were removed while sieving through I.S. sieve. The material passing through above sieve was used to prepare soft soil sub-grade. The thickness of soil sub-grade prepared was more than two times (17 cm) the diameter of the footing. This was to get the pressure bulb to develop fully beneath the footing. The soil, which passes through 2.00mm I.S. sieve, was weighed to the required quantity (40 kg), for the required thickness of the soil sub-grade to be achieved. Water was then added to the soil in the required quantity to make the mixture. 13% water was added (optimum water content, 14%) by volume (5.2 liter). The mixture was then well mixed with hand, so that there were no lumps of soil left and the mix get uniform in color. Before the placing of soil in to the cylindrical tank, at the bottom of it, was placed a filter paper, to check the soil to drain away from the small holes made there.

The soil was filled in the tank to the required thickness in three layers, and each layer was given 55 number of blows with a hammer of self-weight 11.43 kg and having a diameter of 14.0cm. Each time during the placement of soil in the tank, the same number of blows were given, to achieve same density of each layer. After compacting the soil, the top of the sample was made smooth with the help of light blows of the hammer. By this method a density of 20.2kN/m^3 was achieved. Sample was then kept for saturation for seven days, after which the moisture content of the soil reaches up to about 22%, for all the cases. For the load test on the granular fill over soft soil, the soft soil sub-grade was prepared as before and on top of it, the sand was laid using the rainfall technique.

The weight of sand was noted before it was allowed to fall on the soil sub-grade. The sand was poured to the required thickness, equal to the diameter of the footing. The height of fall of sand was maintained at 64 cm for each test to achieve same relative density (56.17%), when desired thickness of sand was reached, the surface of same was leveled. For the tests, with non-woven geotextile as reinforcing material, the same was placed at required depth, at the middle, two-third and one-third depth of sand layer and on top of it the sand was again poured. Diameter of the geotextile was kept equal to the diameter of cylindrical tank.

The footing is kept at the center of the tank, to keep it at twice the distance of its diameter from the tank walls. This is to minimize the effect of walls. The tilting of footing is checked with the help of a bubble tube, but in all the experiment there is little tilting, because the footing is only restrained on two sides. Figs. 3.1 and 3.2 show the sketch of the experimental set up and the geometric parameter of the reinforced model foundation. Vertical load was applied to the footing of diameter 7.5 cm with the help of small loads kept on the platform built to keep the load. The load was kept for 24 hours to achieve full settlement. The settlement corresponding to a particular load was measured with the help of four dial gauges kept at the four corners of the footing. The readings from these dial gauges were taken to calculate the immediate and the final settlement of the footing.

After 24 hours, next increment of the load was applied on the previous one and the same procedure was followed to take the reading.

After the test was completed, a small sample of soil was taken to determine moisture content and the density of the soft soil sub-grade. Afterwards, the soil sample from the tank was removed completely and the tank was prepared for the next set of experiments. The sample was prepared as explained above, and for the test on geotextiles reinforced granular fill-soft soil system, same procedure was repeated with the geosynthetic placed at the required depth.

The results obtained from the tests carried out are presented in chapter 4.

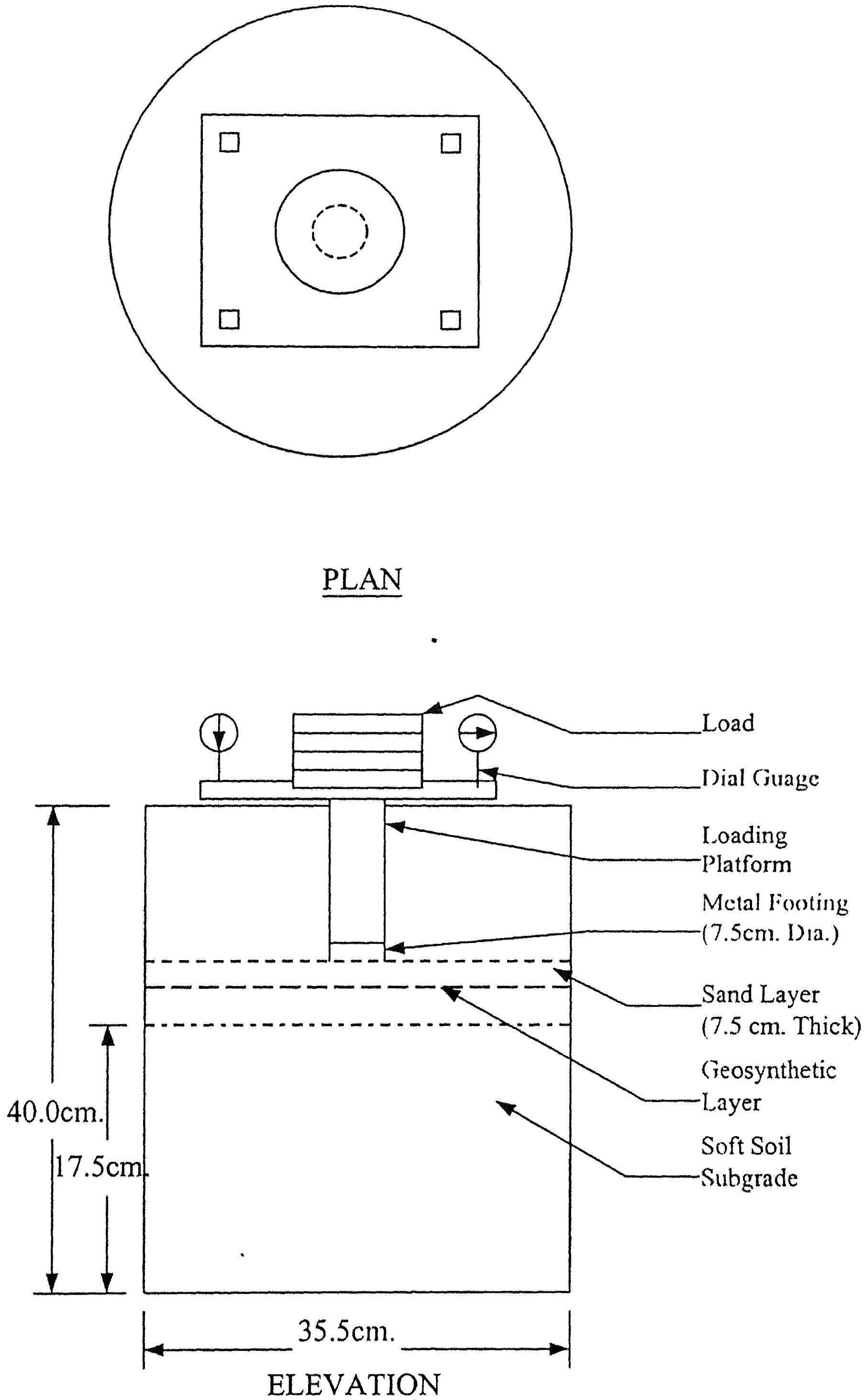

Fig.3.1: MODEL TEST SETUP

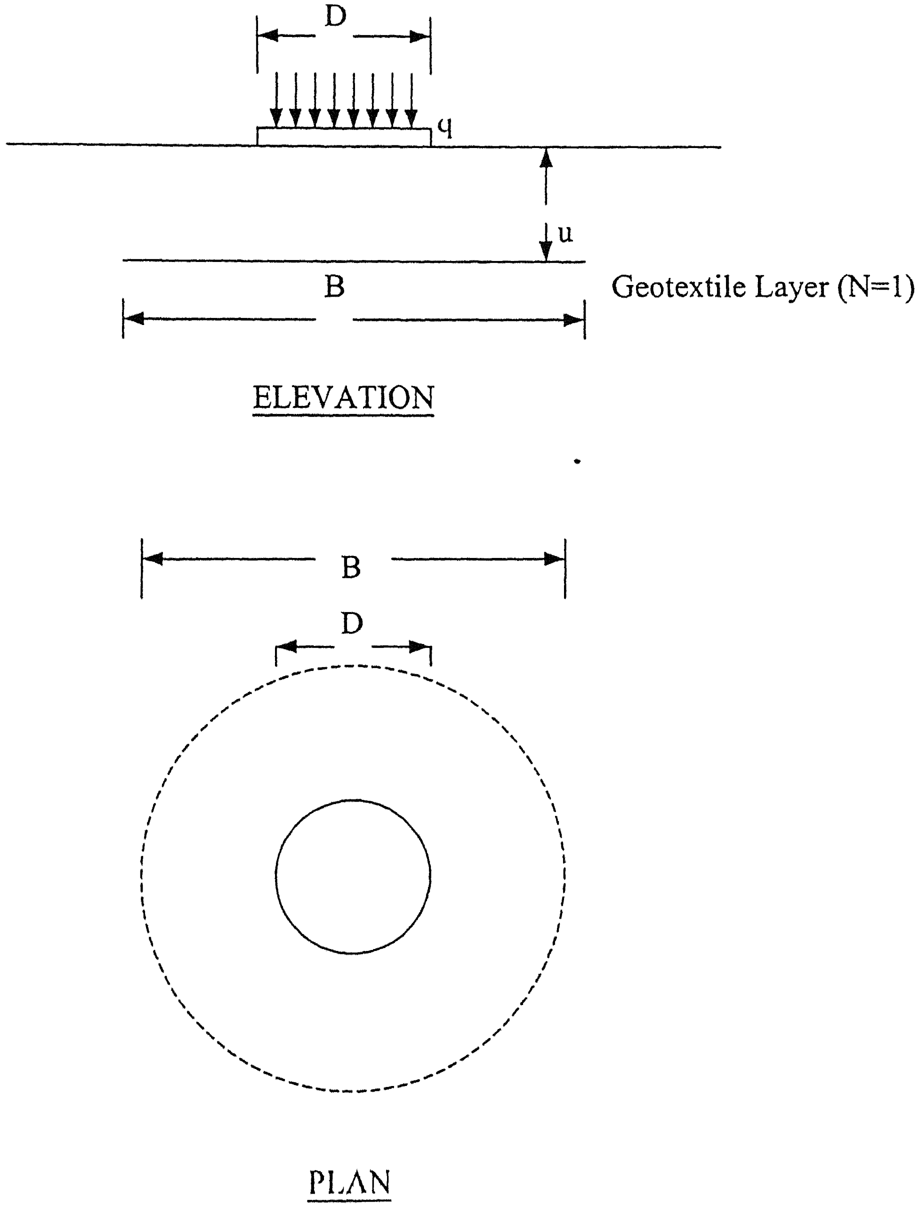

Fig. 3.2: GEOMETRIC PARAMETERS OF REINFORCED FOUNDATION

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

The results of the tests conducted as described in chapter 3 are presented in this chapter. Several tests were conducted on the materials with circular footings, to study the settlement profile of soft soil-granular fill, lying over the soft soil and when a single layer of the reinforcement was placed horizontally below the footing at the middle of the granular fill.

4.2 PROPERTIES OF SOIL

4.2.1 Particle size distribution

The particle size distribution curve of soil used for the model test is shown in Fig. 4.1. From the result it is clear that the soil used is well graded with all types of particles ranging from sand to clay, having a uniformity coefficient (C_U) of 1.45 and coefficient of curvature (C_C) of 0.95. From the grain size distribution curve it is also clear that the soil used in the test contains large sand proportions (69%) and a small amount of clay fraction (7.8%), also there is considerable amount of silt fraction (23.2%). So we can term the soil as silty sand.

4.2.2 Specific gravity

From the tests, the specific gravity of the soil is found to be 2.67.

4.2.3 Density of soil

The average wet and dry density of soil is found to be 20.22kN/m^3 and 16.65kN/m^3 respectively.

4.2.4 Atterberg limits

Tests on the soil samples are conducted to determine the Atterberg limits. From these tests the liquid limit of the soil, used in test is found to be 27.1% and the plastic limit is found to be 18.18%. The plasticity index of the soil used in test is found to be 8.92%.

4.3 PROPERTIES OF SAND

4.3.1 Particle size distribution

The particle size distribution of sand used in the test is shown in Fig. 4.2. From the graph it is clear that the sand used for model test is uniformly graded, having a uniformity coefficient (C_U) of 2.12, and coefficient of curvature (C_C) of 1.04.

4.3.2 Specific gravity

The specific gravity of the sand used in the test is found to be 2.67.

4.3.3 Density of sand

The maximum and minimum density of the sand is observed as 14.67 kN/m^3 and 13.22 kN/m^3. These tests are carried out to determine the relative density of the sand used in the model experiment. The relative density of the sand layer used in the test is found to be 56.17%.

4.3.4 Direct shear test

The direct shear tests are carried out to determine the angle of internal friction and the shear modulus of the sand. The graph between normal stress and shear stress, to determine friction angle is shown in Fig. 4.3. Also the graph between p and q is shown in Fig. 4.4. The results obtained from the tests are given in Table 4.1.

Table 4.1: Physical properties of Sand used in the model test

Uniformity coefficient (C_U)	2.12
Coefficient of curvature (C_C)	1.04
Effective size D_{10} (mm)	0.085
Median size D_{50} (mm)	0.155
Specific Gravity G_s	2.67
Max. dry unit weight $\gamma_{d\ max}$ (kN/m^3)	14.67
Min. dry unit weight $\gamma_{d\ max}$ (kN/m^3)	13.22
Minimum void ratio $e_{min.}$	0.48
Maximum void ratio $e_{max.}$	0.70
Relative Density D_r	56.17%
Young's Modulus of Sand E_s (MN/m^2)	1.628
Angle of Internal Friction (ϕ)	39°

4.4 PROPERTIES OF GEOTEXTILE

4.4.1 Opening size diameter

This test is carried out to measure the pore size/ opening size of the non-woven geotextile, made up of polypropylene and used as the reinforcing material. To carry out this test glass beads of different diameters are taken, ranging from the diameter of 0.075 mm to 0.85mm. These glass beads are passed separately and individually from the geotextile. Each time a known amount of glass beads are poured over the geotextile inside an apparatus used for sieving, and the same is kept for about 20 min in shaking machine. After 20min the amount of material passed from the geotextile is weighed and the results are plotted between percentage finer and beads size. Fig. 4.5 shows the graph between percentage finer and beads size. From the graph the pore size of the geotextile is found to be O_{95} = 0.108mm. O_{90} = 0.125mm., O_{50} = 0.165mm.

4.4.2 Tension test

The main function of the geotextile used in the model test is to act as tensile reinforcement to the soft soil and the granular fill. Hence, to determine the tensile strength of the geotextile, tensile tests are carried out. These tests are carried out in ACMS building, I.I.T. Kanpur. One of the tests is conducted at a chart speed of 10 mm/min and another is at a chart speed of 20 mm/min. In both the tests, the crosshead speed is kept constant at 10 mm/min. The full-scale load applied is 0.2 kN. From the results plotted in Fig. 4.6, the tensile strength of the geotextile is calculated to be equal to 7.0 kN/m. The extension of geotextile at failure is about 100%.

4.4.3 Density and Thickness

The unit weight of geotextile, having a thickness of 2.0 mm used in the model test is found to be 3.5 kN/m^2.

4.4.4 Friction angle between Sand and Geotextile

Modified direct shear tests are conducted on the geotextile and sand samples to determine the friction angle between them. The graph between normal stress and shear stress is shown in Fig. 4.7. From these tests the friction angle between sand and geotextile is found to be 33°. The results of direct shear test to determine friction angle between geotextile and the sand is shown in Table 4.2.

Table 4.2: Determination of friction angle between Geotextile and Sand

Sr. No.	Normal Stress(kN/m^2)	Shear Stress(kN/m^2)	ϕ_{sg}(Degree)
1.	50	38.33	33°
2.	100	73.33	
3.	150	101.67	

4.5 MODEL TEST RESULTS

Load-Settlement curves obtained from the model test on soft soil, granular fill, soft soil with granular fill and on the geosynthetic reinforced granular fill-soft soil, with the reinforcement placed at one third, middle and at two third depth of the thickness of the granular fill are presented in Figs.4.8 to 4.13.

Fig. 4.8 shows the variation of initial and final settlement with the load of the footing. The settlement of the footing initially is linear up to a stress level of 30.69 kN/m^2 and after this stress the settlement of the footing increases more rapidly and is non-linear. The final settlement also shows same type of variation as the initial settlement. The variation of final settlement is more rapid than the initial settlement. From the figure it is clear that as the load increases, settlement goes on increasing.

Fig. 4.9 shows the variation of settlement of the footing with the application of load on the sand sample alone. In the case of footings on sand, almost all the settlement takes place immediately, hence the curve shows only the immediate settlement of the footing occurred with the application of the load. From the curve it is clear that the settlement of the footing on sand is almost linear with stress.

Fig. 4.10 shows the settlement of the footing with the application of the load. In this case sand layer is provided over the soft soil sub-grade. The thickness of the sand layer is equal to the diameter of the footing. From the figure it is clear that the settlement in this case is less as compared with the settlement of the footing resting directly over the soft soil sub-grade. This is due to the fact that, the introduction of sand layer spreads the load in larger area over the soft soil sub-grade, therefore the stresses acting on the soft soil is less, and hence the settlement. Here also the settlement is linear up to a stress level of about 20 kN/m^2 and after that it becomes non-linear. The rate of settlement goes on decreasing with the increase in load. The overall settlement in this case is less as compared with the case when the footing is resting directly over the soft soil sub-grade.

Figs. 4.11 to 4.13 shows the settlement of the footing when the sand layer is reinforced with a single layer of non-woven geotextile. The reinforcement is placed at two third, half and one third of the depth of the thickness of the granular fill. With the introduction of the geotextile the settlement of the footing further reduces. The rate of settlement also decreases with the increase in the stress level. The settlement for the case when the granular fill is reinforced with non-woven geotextile, is less as compared to the situation when the granular fill was not reinforced. This may be due to the fact that the geotextile also takes some of the stresses coming from the footing and stretches in this process.

From the above figures, it is observed that the load bearing capacity of soft soil alone is lowest followed by the load bearing capacity of the granular fill alone. For the case when granular fill is placed on top of soft soil, without any reinforcement, the load bearing capacity of the system increases and the settlement as a whole reduces when compared with the soft sub-grade alone. Now, for the case when geotextile is introduced as a reinforcing material the settlement is reduced and the reduction in the settlement is maximum when the geotextile is placed at the middle of the granular fill, than other two cases of the reinforced granular fill. It can be observed from Figs. 4.12 and 4.13 that the immediate settlement reduction is about 7% while the final settlement reduction is about 5% at a stress level of 25kN/m^2. For a stress level of 40kN/m^2 the immediate and final settlement reductions are of the order of about 9% and 21% respectively.

Comparing the settlements of the footing from Figs. 4.8 and 4.11, one can see that the footing settles less when the granular fill is reinforced with geotextile. Fig. 4.11 shows the case when the footing is resting over reinforced granular fill and the reinforcement is placed at a depth of 2/3rd of the thickness of the granular fill. In this case the reduction in the immediate and final settlement is of the order of about 26% and 23% at stress of 25kN/m^2 and about 17% and 31% at a stress level of 40kN/m^2.

Fig. 4.12 shows the variation of the settlement in case of the reinforced granular fill, when the reinforcement is placed at a depth of 1/2 the thickness of the granular fill. From comparing it with Fig. 4.8, it can be observed that there is a reduction in the immediate and final settlement of about 34% at a stress level of 25kN/m^2 and the corresponding reduction is about 40% and 44% at a stress level of 40 kN/m^2.

Fig. 4.13 is the case when the geotextile is placed at 1/3rd depth of the thickness of the granular fill. The immediate and final settlements are reduced to about 28% and 25% respectively at a stress level of 25kN/m^2. The corresponding reduction is of the order of about 37% and 34% respectively at a stress level of 40 kN/m^2.

From Figs. 4.8 to 4.13, it can be observed that the reduction in the immediate settlement is of the order of about 23%, 30% and 20% for the cases when reinforcement is present at 1/3rd, 1/2 and 2/3rd depths of the granular fill thickness. The reduction in the final settlement is of the order of about 25%, 34% and 5% respectively for the same cases at a stress level of 25kN/m^2. For a stress level of 40kN/m^2 the reduction in the immediate and final settlement is of the order of about 29%, 33% & 9% and about 18%, 29% and 12% respectively.

From all these figures it is clear that with the introduction of the geotextile the settlement of the footing reduces. The reduction in the settlement is maximum for the case when the geotextile is placed at half the thickness of the granular fill. The maximum reduction in the immediate and final settlement is of the order of 34% at a stress level of 25kN/m^2. The settlement is more about 13% and 9% when reinforcement is placed at 1/3rd and 2/3rd depth in the granular fill than the case when the reinforcement is placed at 1/2 the depth.

Figs. 4.14 to 4.16 shows the plots for both the densities, for soft soil subgrade, soft soil with unreinforced granular bed and soft soil with reinforced granular bed with the geotextile layer placed at 2/3rd depth of the granular fill. From Fig. 4.14 it is clear that the immediate and final settlement of the footing resting on the soft soil of density 19.12kN/m^3 is 58.2% and 46.8% more than the case when footing is on the soft soil subgrade of density 20.53kN/m^3. Fig. 4.15 shows the comparison between the settlements of the footing when the granular fill is placed over the soft soil subgrade. In this case also the immediate and final settlement of the footing is more about 31% and 27.60% for the situation when density of soft soil is about 19.22 kN/m^3, than when the density is 20.32 kN/m^3.

Fig.4.16 shows the situation when the granular fill, placed over the soft soil is reinforced and the reinforcement is provided at 2/3rd depth of the granular fill. Initially, the immediate and final settlement of the footing is more when the density of soft soil is 20.15 kN/m^3 as compared to the situation when the density of soft soil is 18.98 kN/m^3, but after the stress value of about 40kN/m^3, the settlement becomes less for the case when the density of soft soil is more.

4.6 BEARING PRESSURE RATIO

The bearing pressure ratio (BPR), defined as the ratio of the bearing pressure of reinforced soil for a particular settlement to that of unreinforced soil for the same settlement.

$$\mathbf{BPR} = q_a / q_{ao}$$

Where q_a = bearing pressure of reinforced soil for a given settlement and

q_{ao} = bearing pressure of unreinforced soil for same settlement.

The values of bearing pressure ratio obtained from the tests at a settlement value of 3.0 mm of the footing is presented in the Table 4.3. To show the effect of the depth of first layer of reinforcement, on the bearing pressure ratio, a plot of BPR vs. non-dimensional parameter u/D is plotted as shown in Figs. 4.17 and 4.18. The figure shows that BPR first increases as the geosynthetic reinforcement is placed from $1/3^{rd}$ to 1/2 the depth of granular layer and then decreases as the same is moved from 1/2 depth to $2/3^{rd}$ depth of the granular fill layer.

In Fig. 4.17 for the reinforcement provided at depth 1/3D, 1/2D and 2/3D in the granular fill (D is the diameter of the footing), the BPR values for the footing are 1.40, 1.93 and 1.31 (For immediate settlement) and 1.48, 2.05 and 1.40 (For final settlement) respectively. These BPR values are for the case of soft soil and soft soil with reinforced granular fill.

For the case of soft soil with unreinforced granular fill and soft soil with reinforced granular fill (Fig. 4.18), the BPR values obtained are 1.24, 1.71 and 1.16 (For immediate settlement), and 1.34, 1.85 and 1.22 (For final settlement). These are for the case when the geotextile is placed at a depth of 1/3D, 1/2D and 2/3D in the granular fill. For the case of soft soil and soft soil with unreinforced granular fill, the value of BPR is found to be 1.13 (For immediate settlement) and 1.11 (For final settlement) of the footing. Similar observations were made by Guido (1986), Binquet and Lee (1975) and others.

Plots for immediate and final settlements of the footing with the depth of reinforcement layer are shown in Figs. 4.19 and 4.20. These curves are plotted for three stress values equal to $10kN/m^2$, $30kN/m^2$ and $50kN/m^2$. From these figures, it is observed that both immediate and final settlements of the footing go on increasing as the depth of reinforcement increases and is maximum when the geotextile is laid at a depth of 2/3D in the granular fill.

Fig. 4.21 shows the comparison between the settlement of the footing resting on the reinforced granular fill-soft soil, from the analytical studies carried out by Shukla and Chandra (1994) and from the present experimental study. The reinforcement is placed at the middle of granular fill in both the cases. The load versus settlement curves obtained from the theoretical study matches well with the experimental results for low stresses, but the observed settlement is considerably less than that calculated from the analytical study for higher stresses.

The generalized conclusions drawn from the experimental studies were presented in the next chapter.

Table 4.3: Effect of depth of reinforcement on the BPR for Circular Footing

<table>
<tr><td colspan="2">Soft Soil
q_o (kN/m^2)</td><td colspan="3">Reinforced Granular Fill Soft Soil System
q_{ars} (kN/m^2)</td><td colspan="2">BPR
(q_{ars}/q_o)</td><td colspan="2">BPR
(q_{as}/q_o)</td></tr>
<tr><td>I</td><td>F</td><td rowspan="2">Geotextile at 1/3rd depth</td><td>I</td><td>F</td><td>I</td><td>F</td><td>I</td><td>F</td></tr>
<tr><td rowspan="3">34.15</td><td rowspan="3">26.31</td><td>47.70</td><td>39.00</td><td>1.40</td><td>1.48</td><td rowspan="8">1.13</td><td rowspan="8">1.11</td></tr>
<tr><td>Geotextile at half depth</td><td>66.00</td><td>53.88</td><td>1.93</td><td>2.05</td></tr>
<tr><td>Geotextile at 2/3rd depth</td><td>44.63</td><td>35.63</td><td>1.31</td><td>1.35</td></tr>
<tr><td colspan="2" rowspan="2">Granular Fill + Soft Soil q_{as} (kN/m^2)</td><td rowspan="2">Geotextile at 1/3rd depth</td><td rowspan="2">47.75</td><td rowspan="2">39.00</td><td colspan="2">BPR
(q_{ars}/q_{as})</td></tr>
<tr><td>1.24</td><td>1.34</td></tr>
<tr><td>I</td><td>F</td><td rowspan="2">Geotextile at half depth</td><td rowspan="2">66.00</td><td rowspan="2">53.88</td><td rowspan="2">1.71</td><td rowspan="2">1.85</td></tr>
<tr><td rowspan="2">38.54</td><td rowspan="2">29.11</td></tr>
<tr><td>Geotextile at 2/3rd depth</td><td>44.63</td><td>35.63</td><td>1.16</td><td>1.22</td></tr>
</table>

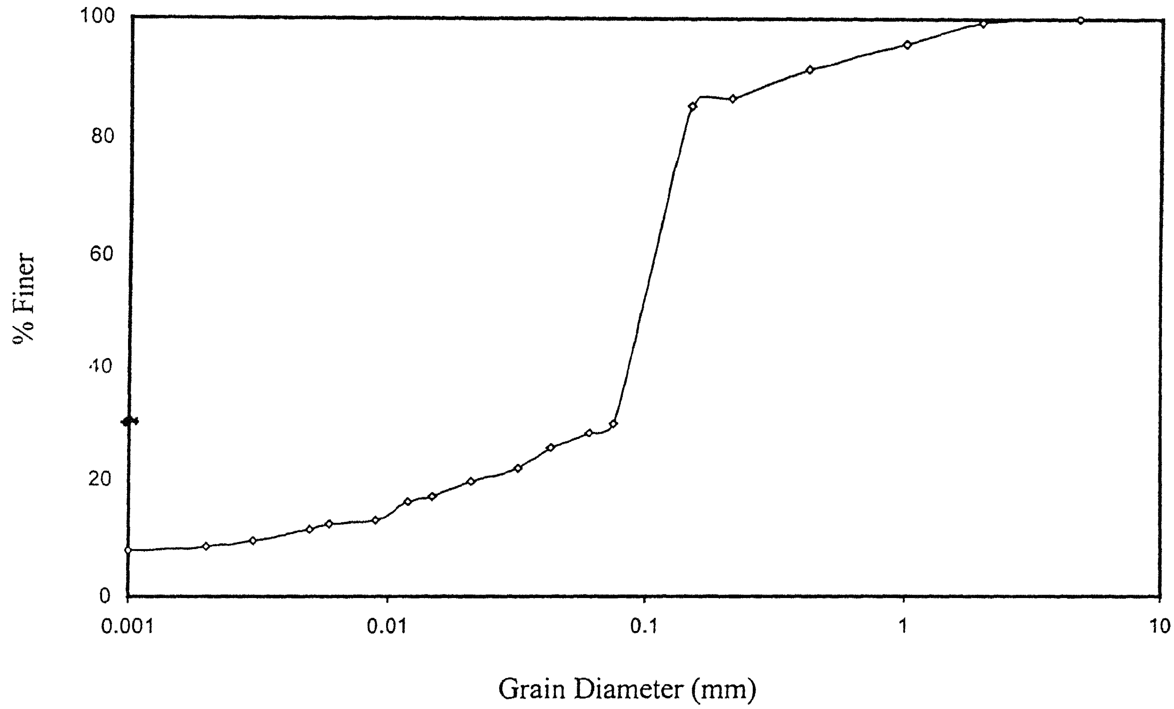

Fig. 4.1: Grain Size Analysis of Soil used in the Test

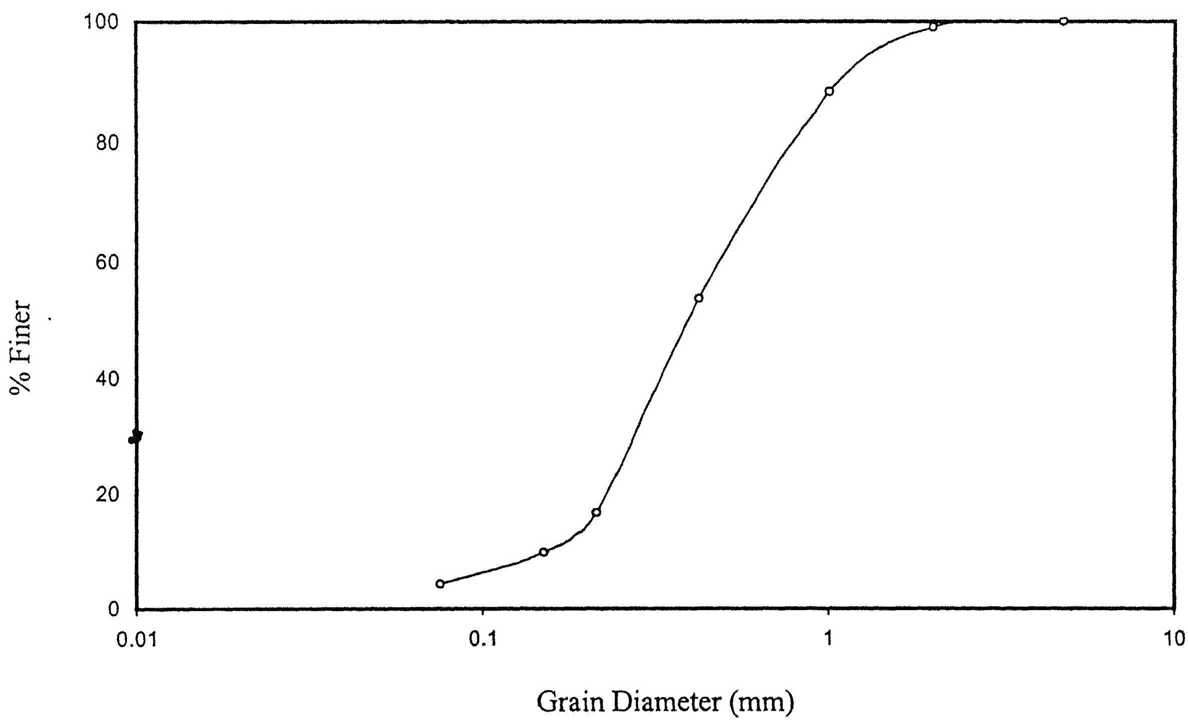

Fig. 4.2: Grain Size Analysis of Sand used in the Test

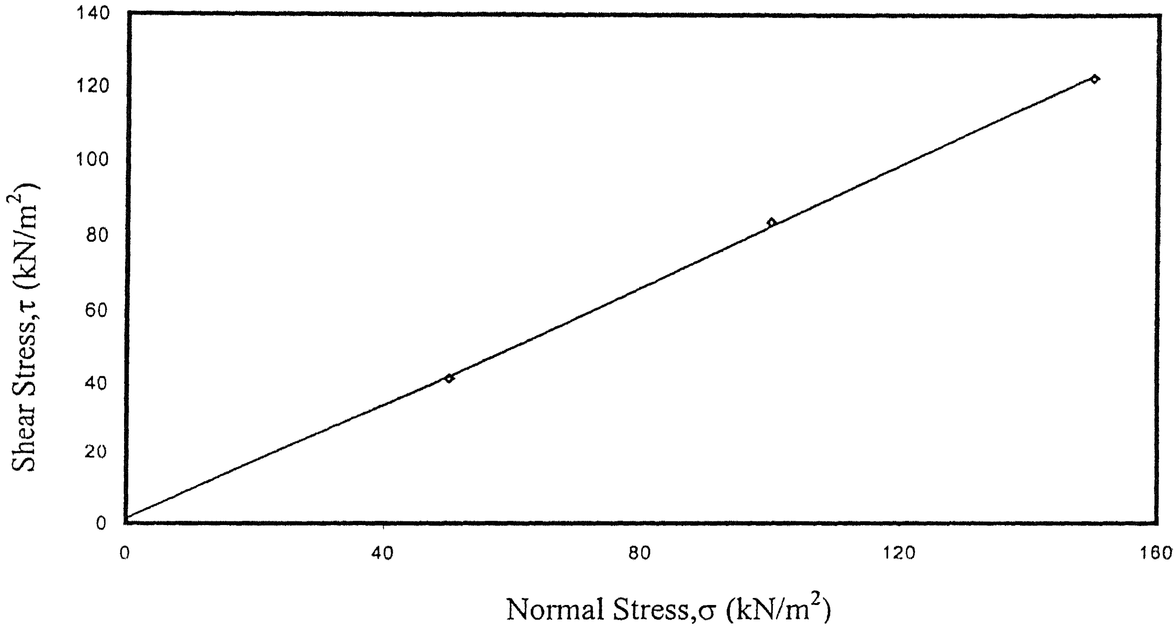

Fig. 4.3: Normal Stress vs. Shear Stress curve for Sand

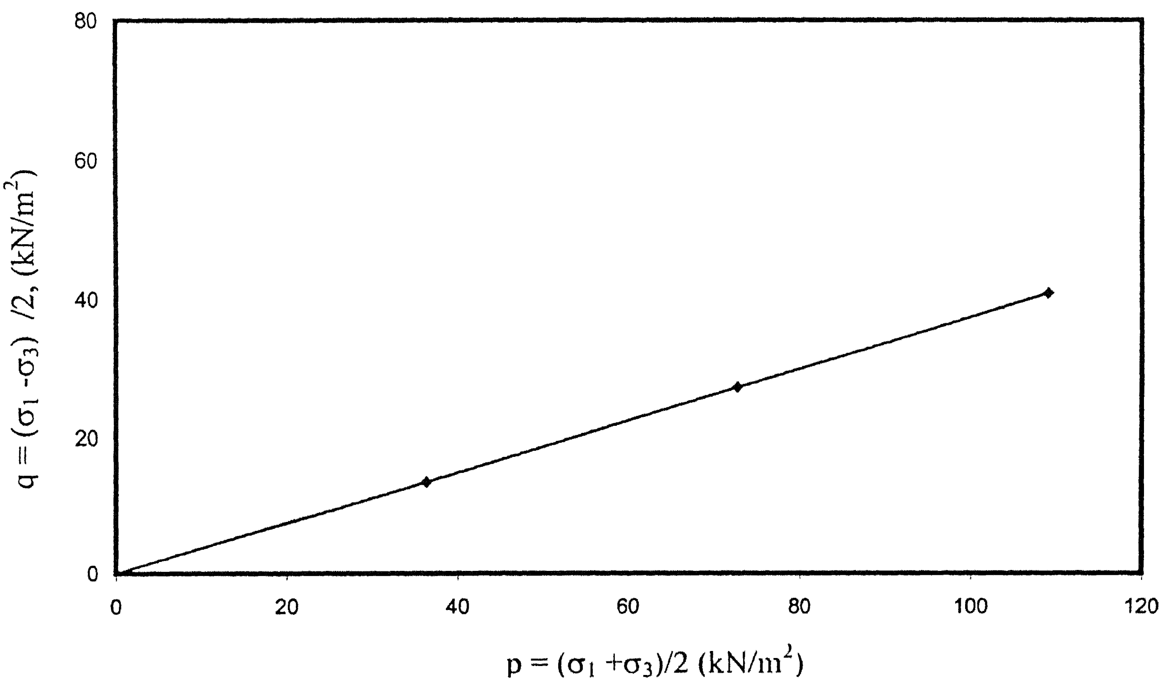

Fig. 4.4: Stress Path (p-q curve) for Direct Shear Test on Sand

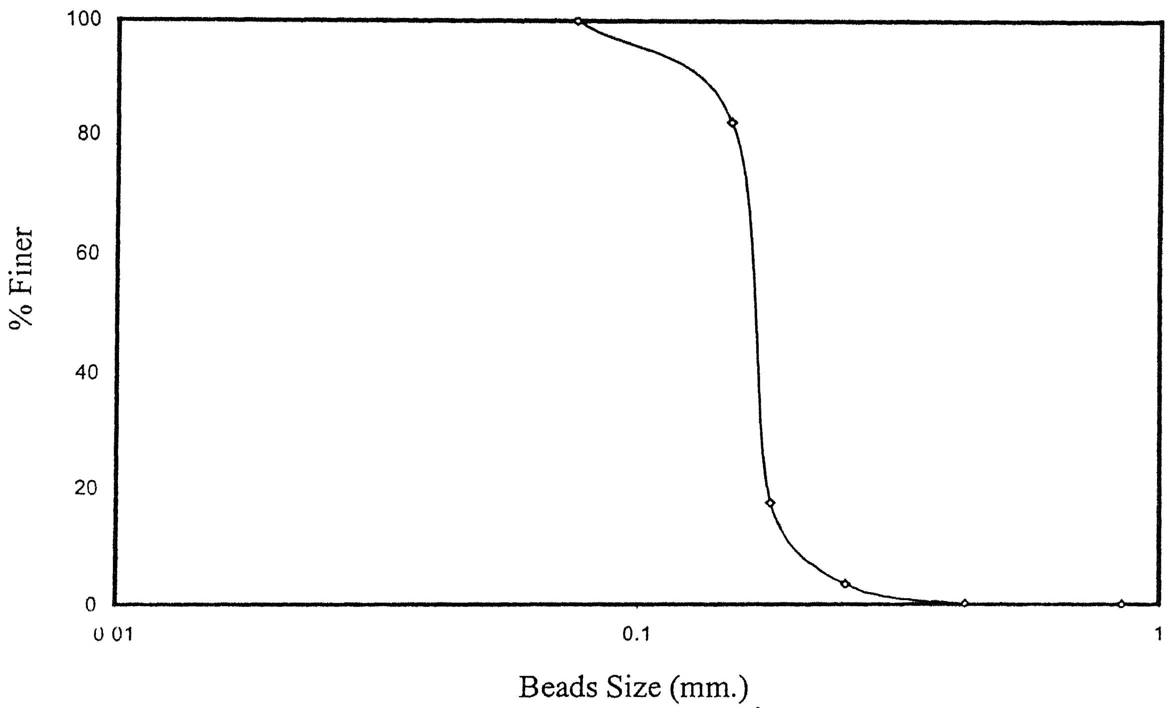

Fig. 4.5: Curve showing the Pore Size Opening of Geotextile

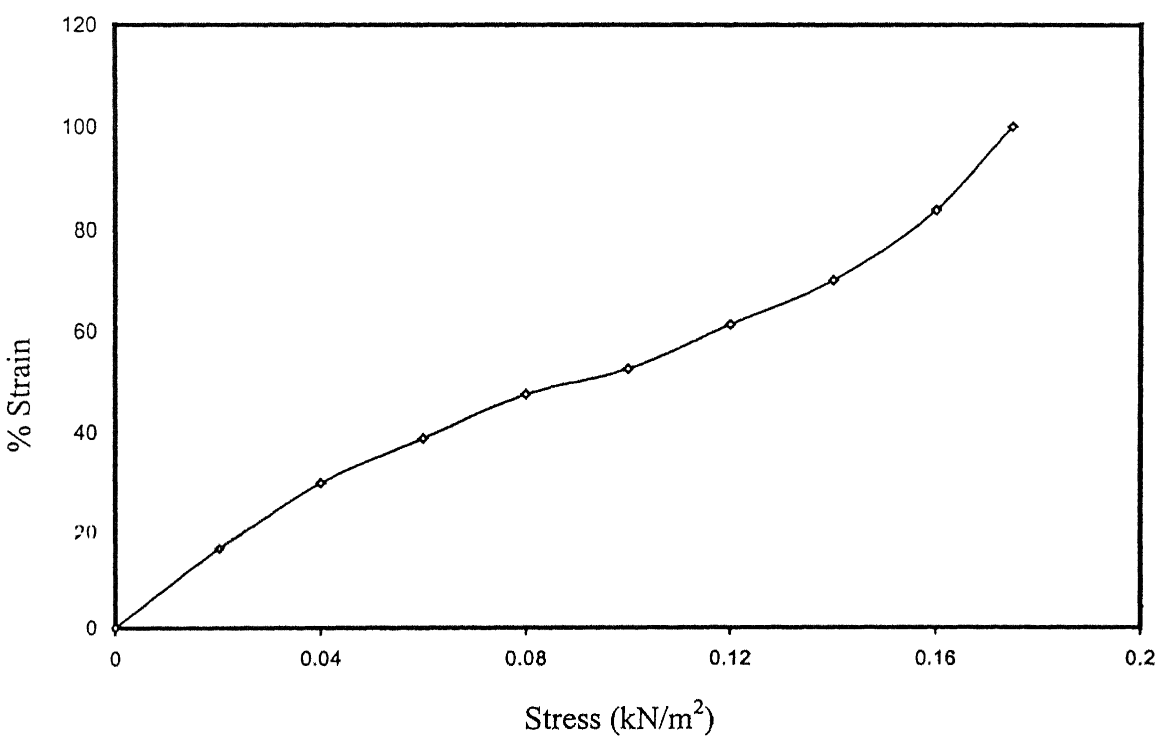

Fig. 4.6: Tension test (Stress-Strain curve) on geotextile

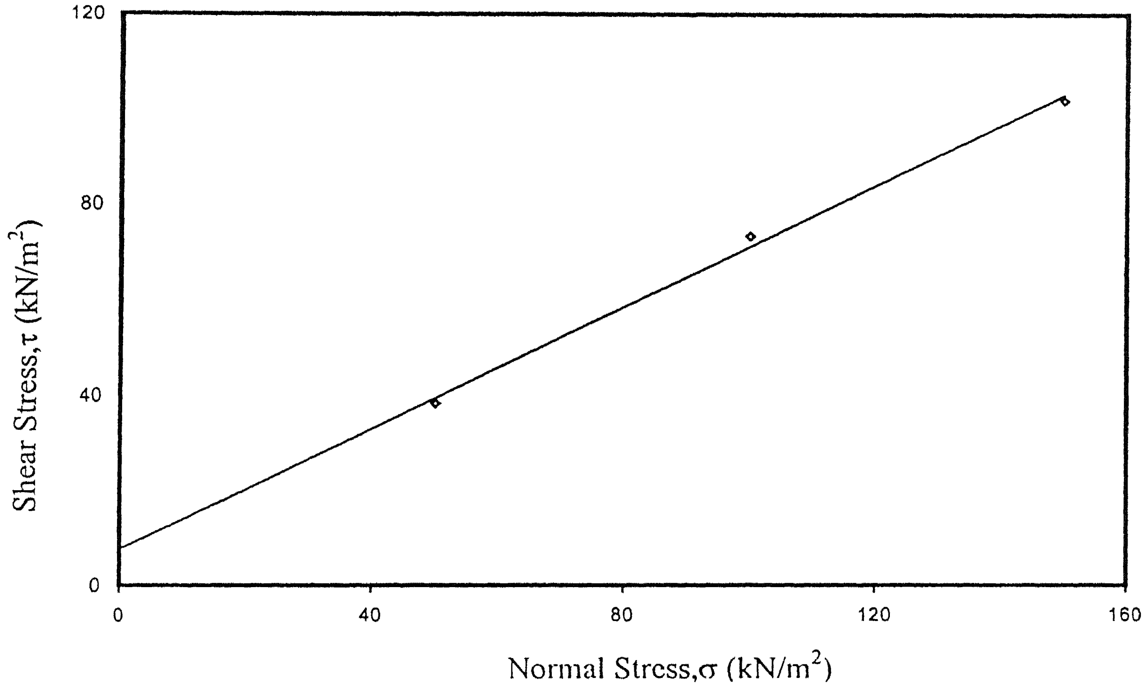

Fig. 4.7: Normal Stress vs. Shear Stress plot for Sand and Geotextile

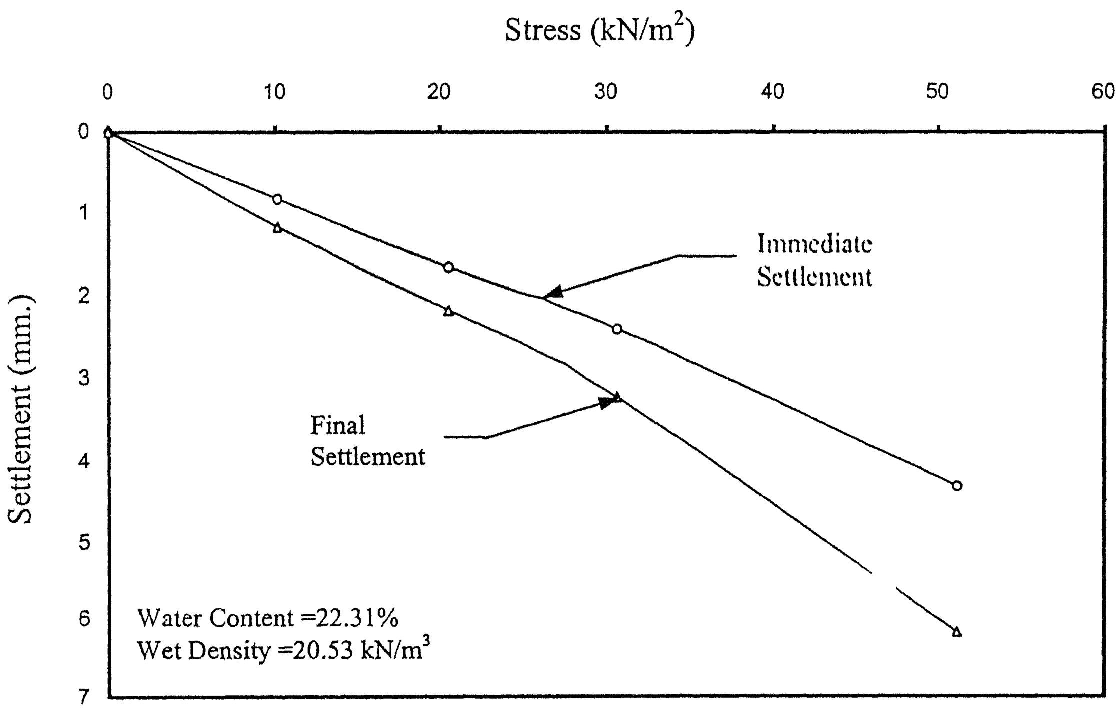

Fig. 4.8: Immediate and Final Settlement of the Footing on Soft Soil

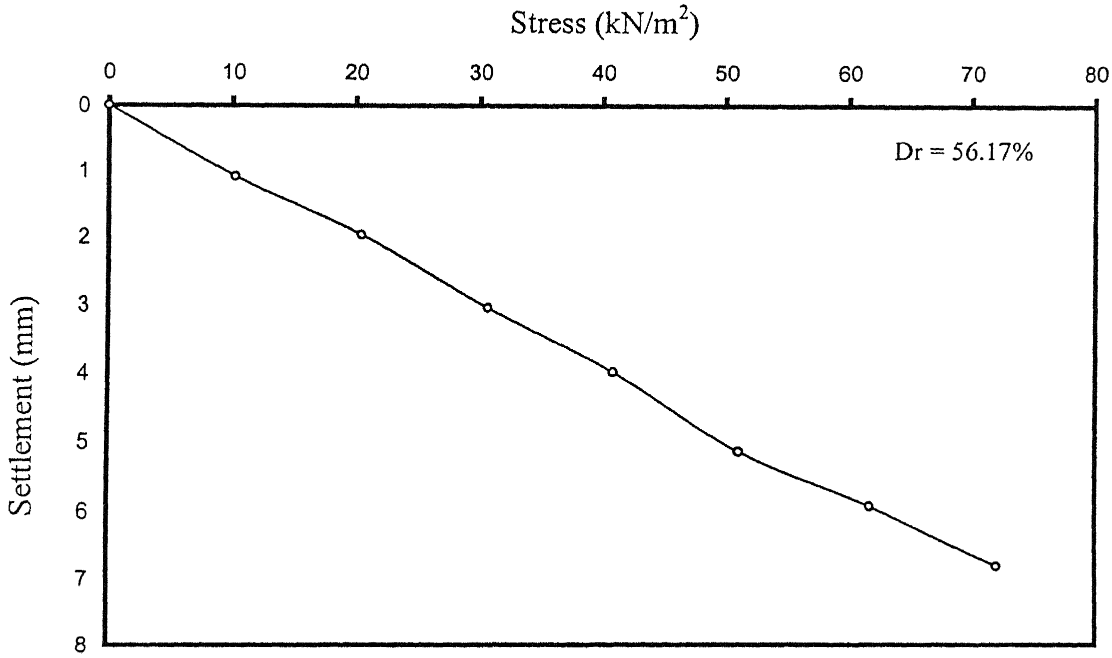

Fig. 4.9: Immediate settlement of Footing on Sand

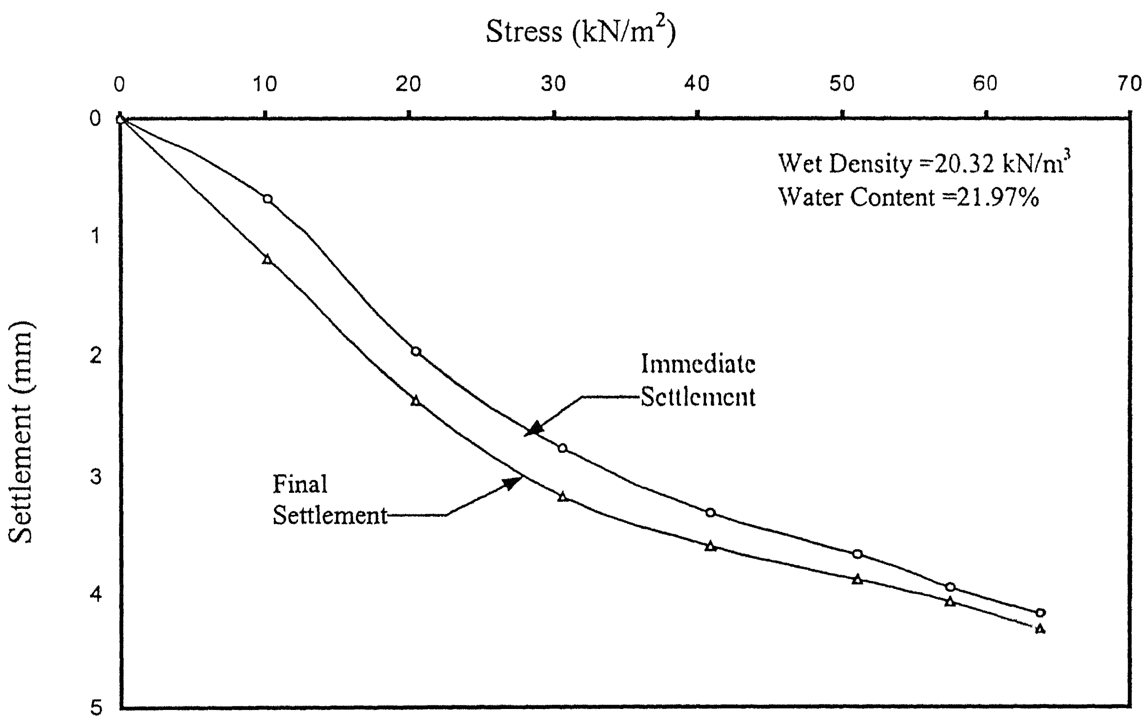

Fig. 4.10: Immediate and Final Settlement of Footing on Soft Soil with Granular Fill.

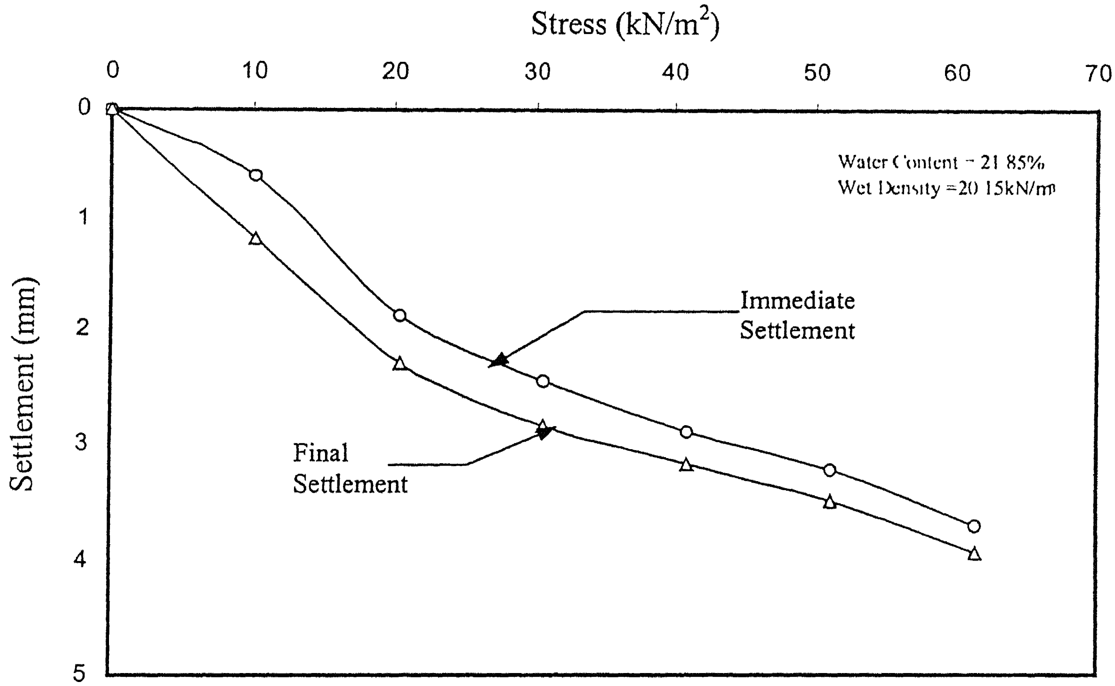

Fig. 4.11 Immediate and Final Settlement of Footing on Soft Soil with Reinforced Granular Fill (Reinforcement at 2/3rd depth of footing)

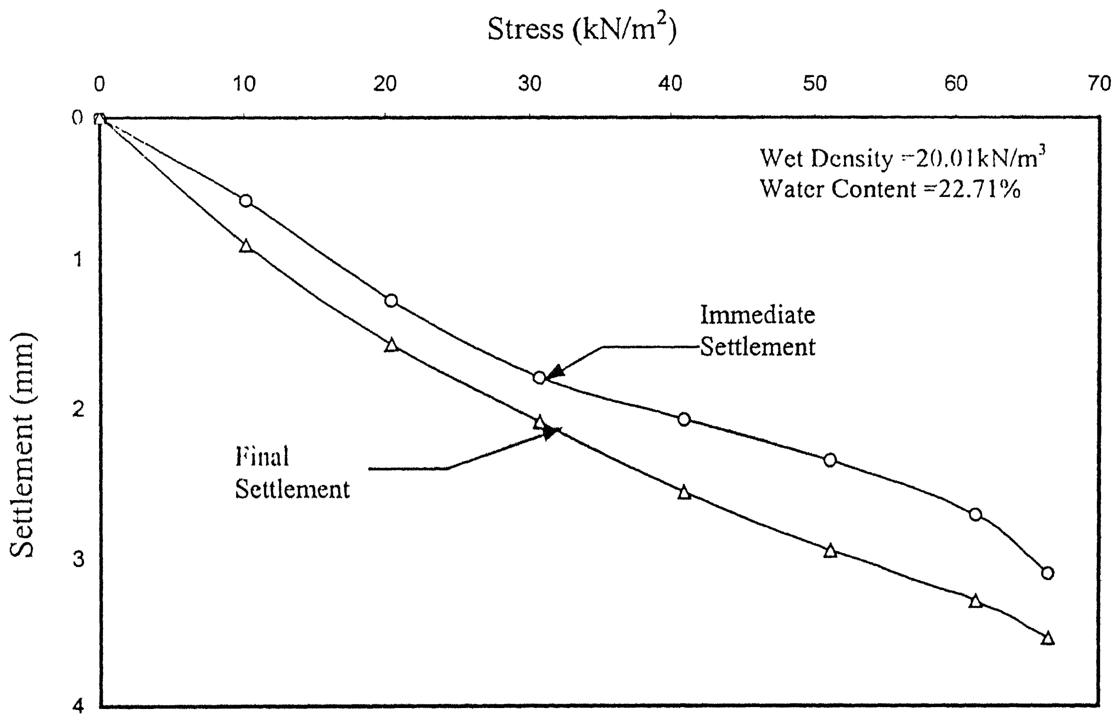

Fig. 4.12 Immediate and Final Settlement of Footing on Soft Soil with Reinforced Granular Fill (Reinforcement at ½ the depth of footing)

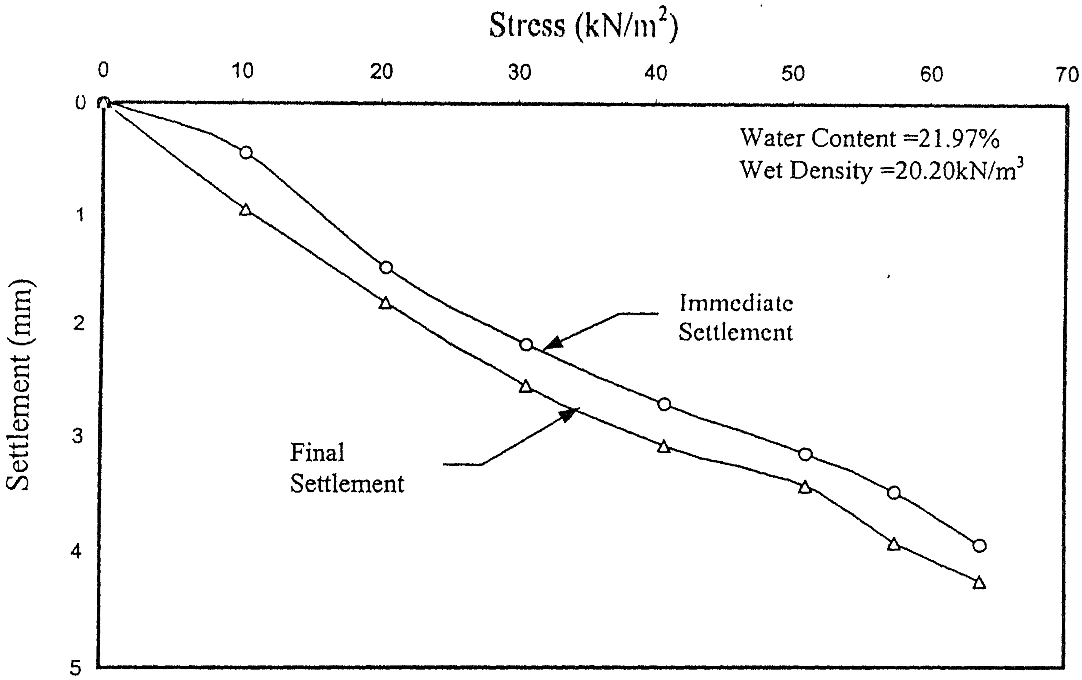

Fig. 4.13: immediate and Final Settlement of Footing on Soft Soil with Reinforced Granular Fill (Reinforcement at 1/3rd depth of footing)

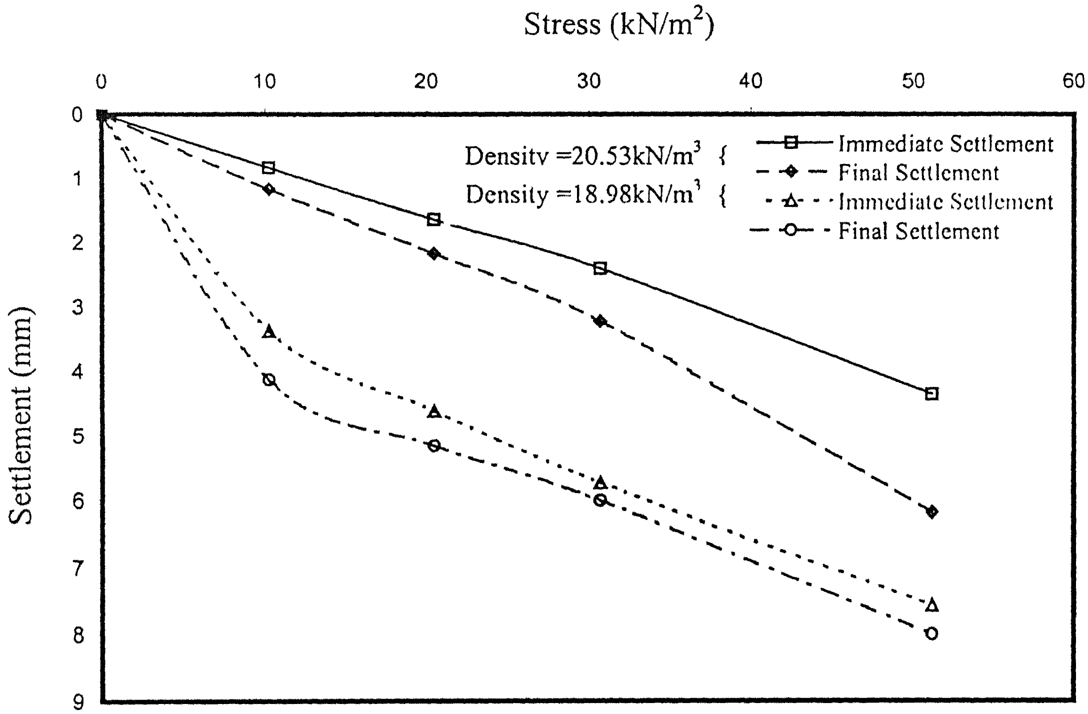

Fig. 4.14: Immediate and Final Settlement of Footing on Soft Soil having different densities.

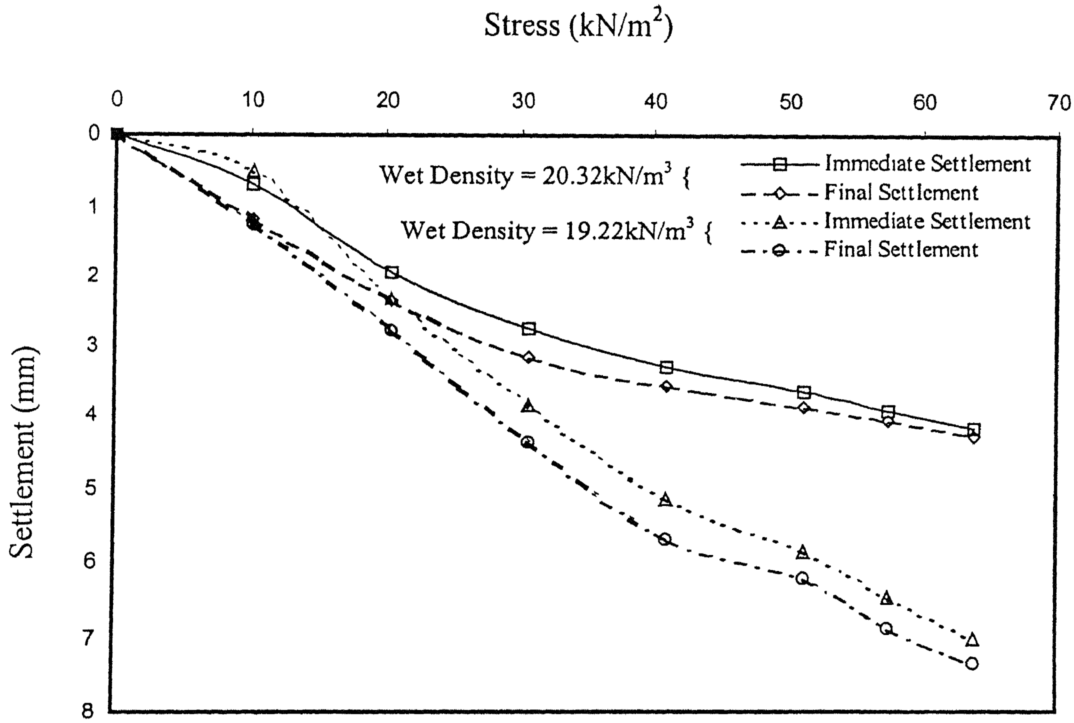

Fig. 4.15: Immediate and Final Settlement of Footing on Soft Soil having different densities with unreinforced granular fill.

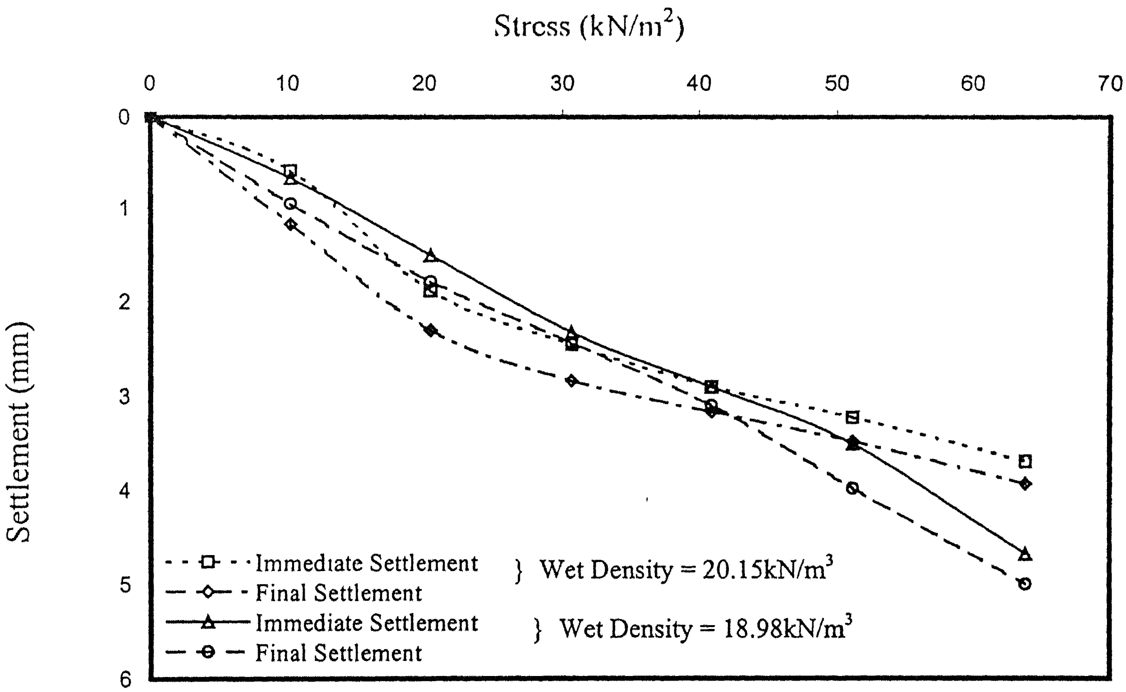

Fig. 4.16: Immediate and Final Settlement of Footing on Soft Soil having different densities with reinforced granular fill.

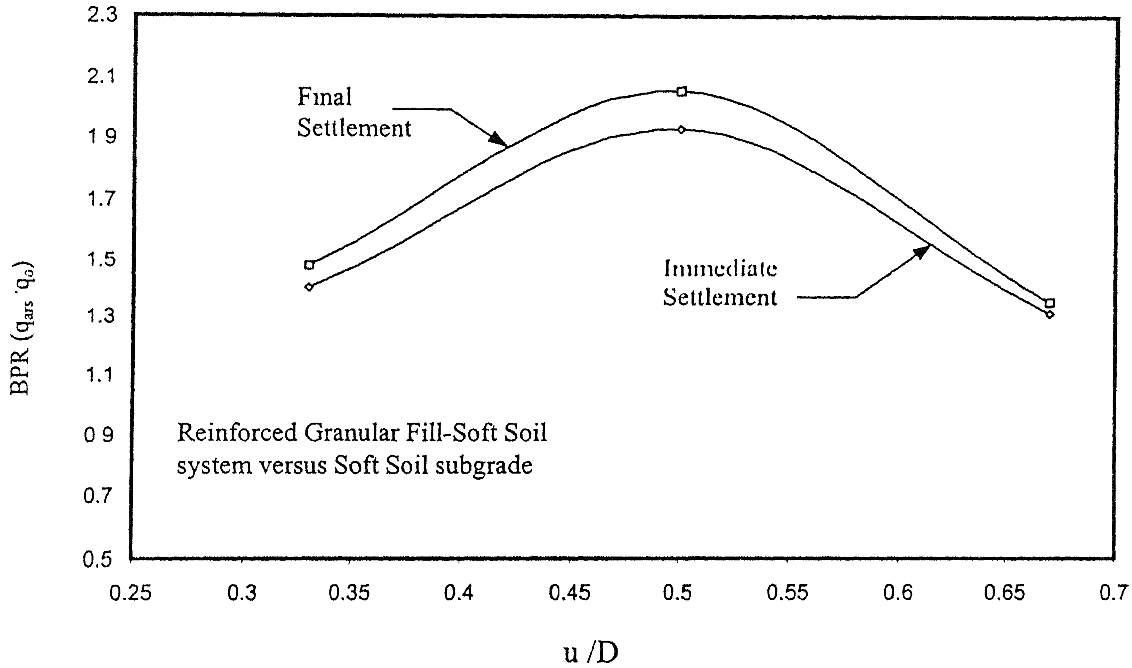

Fig. 4.17: Variation of BPR with the Depth of Reinforcement layer

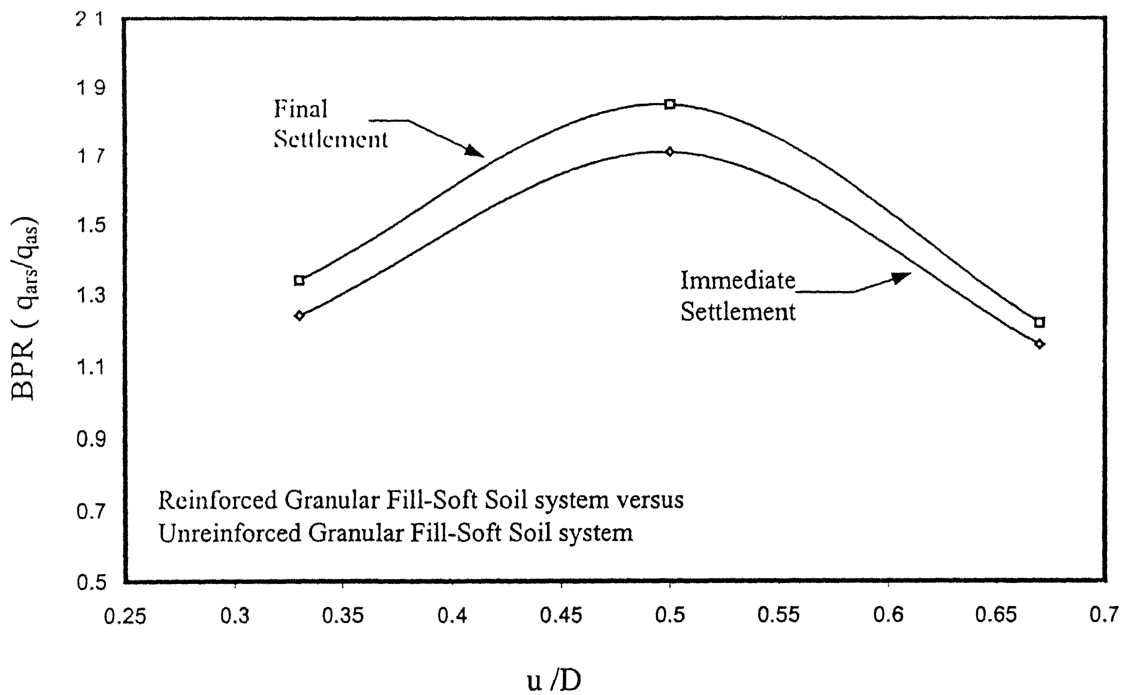

Fig. 4.18: Variation of BPR with the Depth of Reinforcement layer

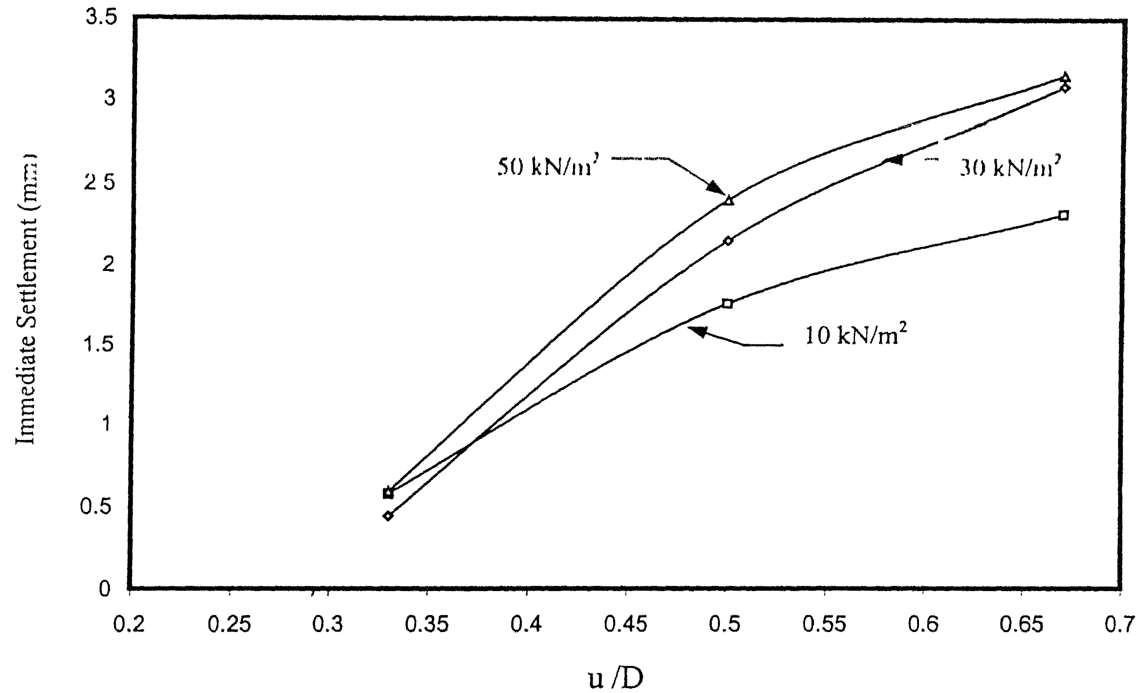

Fig. 4.19: Immediate Settlement of Footing with the Depth of Reinforcement layer.

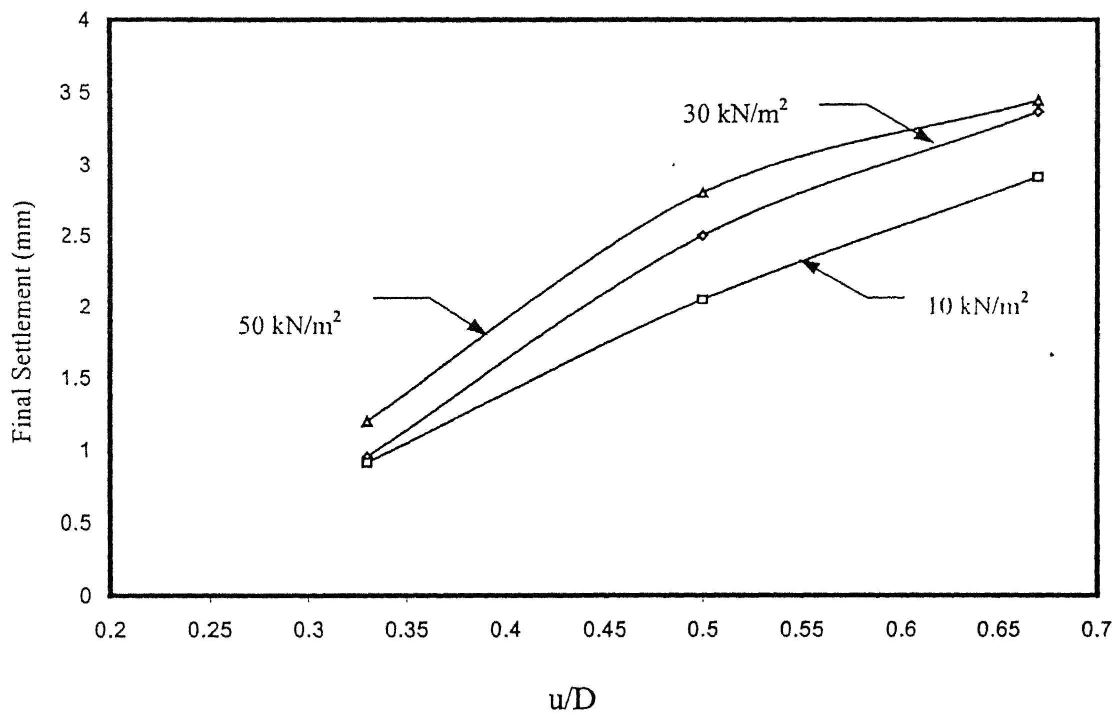

Fig. 4.20: Final Settlement of Footing with the Depth of Reinforcement layer.

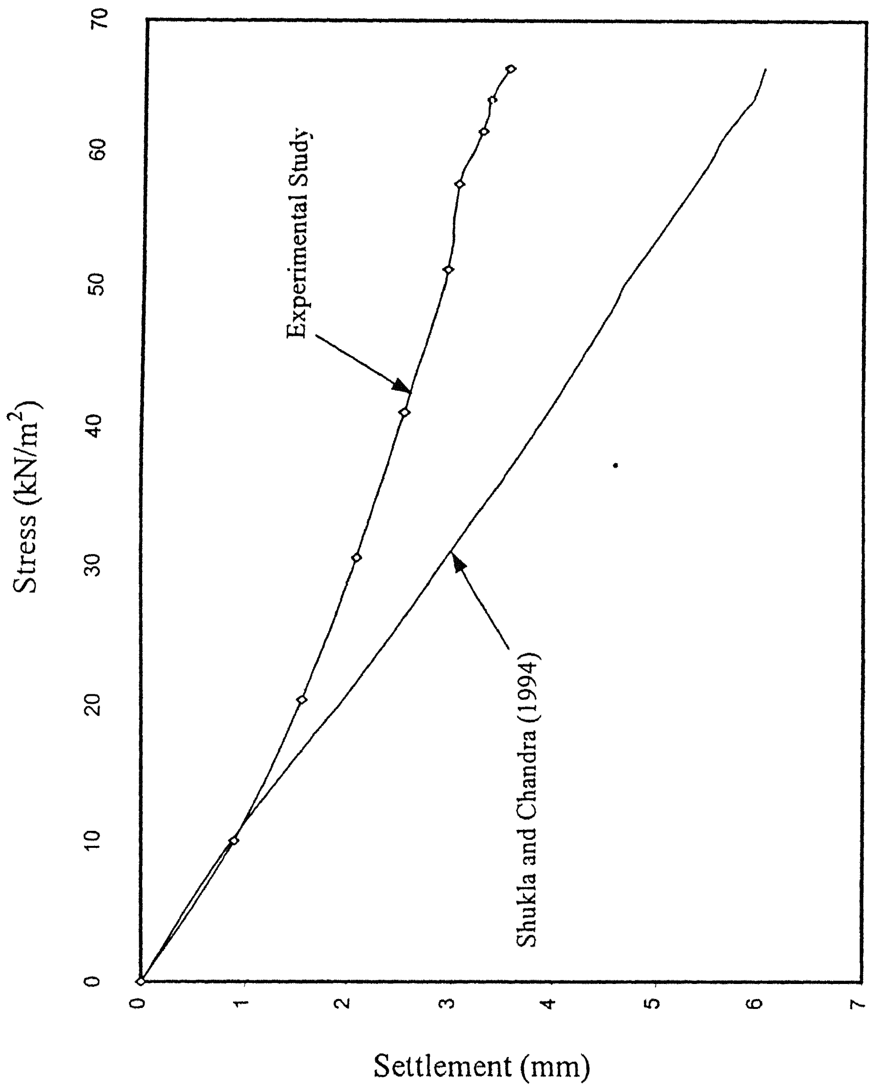

Fig. 4.21: Comparison between Experimental study and numerical study

4. With a little variation in the density of the soft soil subgrade, there is considerable increase in the settlement due to the application of the load.

5. For the soft soil having density of 19.12 kN/m^3, the immediate and final settlements are about 58% and 47%more than for the soft soil having density of 20.53 kN/m^3.

6. The bearing pressure ratio (BPR) of granular fill-soft soil system, first increases as the reinforcement is placed from 1/3D to 1/2D depth, and then decreases as it is placed from 1/2D to 2/3D depth in the granular fill.

7. The bearing pressure ratio (BPR) is maximum, 2.05 for the case when reinforcement is placed at half the depth for immediate settlement and the corresponding value for final settlement is 1.85.

8. The settlement of the footing increases with the depth of first layer of reinforcement for the same load and is maximum for the case when geotextile is placed at 2/3rd depth in the granular fill.

9. For the reinforced granular fill-soft soil, there is a considerable reduction in the settlement of the footing at higher stresses.

10. The observed results agree very well for small loads with the theoretically predicted results.

SCOPE FOR FUTURE STUDIES

The present study takes in to account the variation in the density of the soft soil subgade only. There are various other parameters, which need their evaluation for the better understanding of the behavior of geosynthetic reinforced granular fill-soft soil system. Some of the works that may be carried out in future are:

- Conducting tests by varying the type of geotextile for reinforcement purpose.
- Performing the tests at different relative densities of the granular fill.
- Prestressing the geosynthetic used for reinforcing purpose.
- Conducting tests by using multiple layers of geotextiles as reinforcement.

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